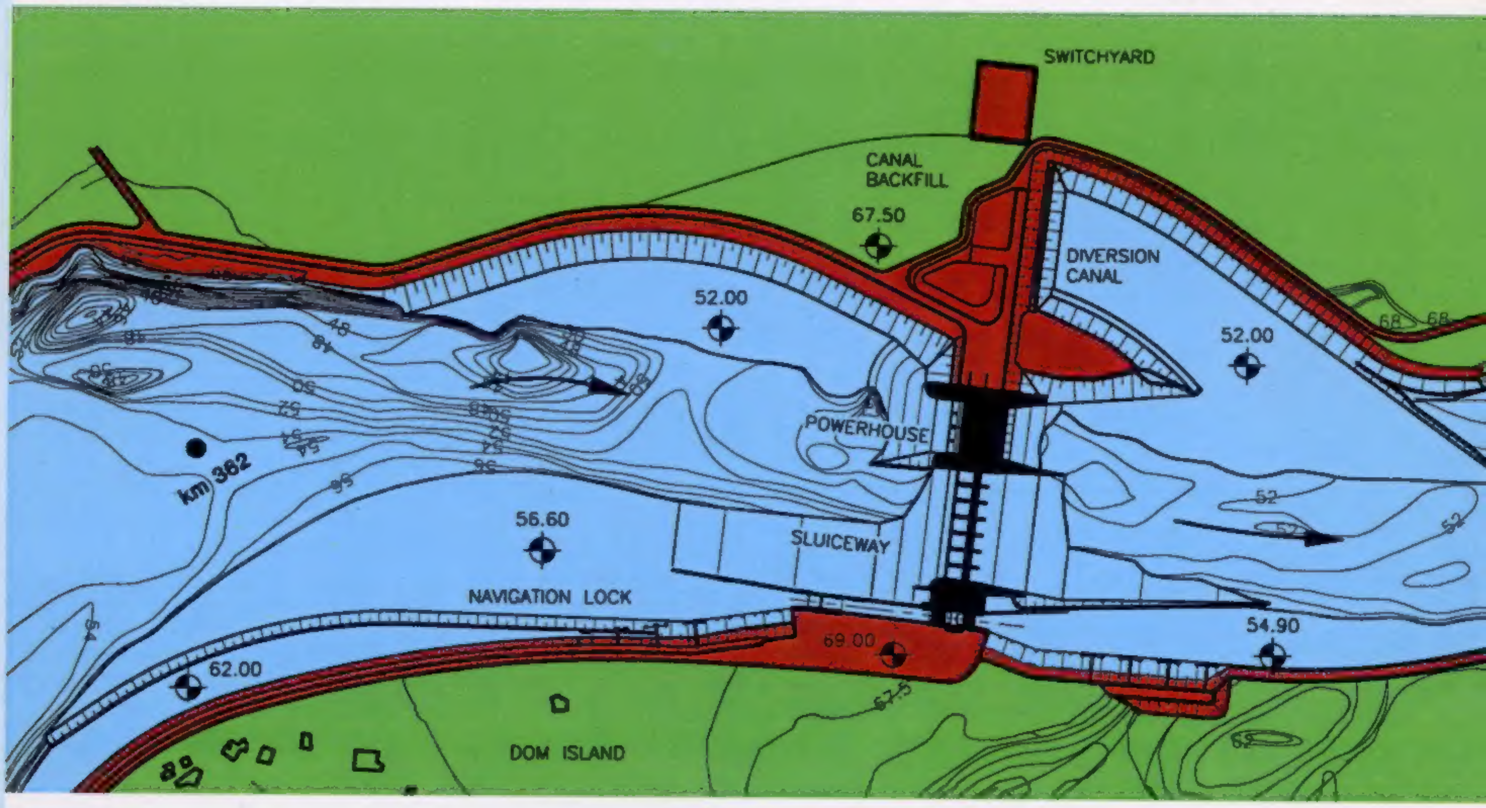
ARAB REPUBLIC OF EGYPT
MINISTRY OF PUBLIC WORKS AND WATER RESOURCES
RESERVOIRS AND GRAND BARRAGES SECTOR

# Naga Hammadi Barrage Development

# FEASIBILITY STUDY
# FINAL REPORT

**AUGUST 1997**




VOLUME 1

MAIN REPORT

## NAGA HAMMADI BARRAGE DEVELOPMENT CONSULTANTS

Consortium Lahmeyer Electrowatt Sogreah

In association with
Arab Consulting Engineers
Moharram - Bakhoum
Egypt

ARAB REPUBLIC OF EGYPT
MINISTRY OF PUBLIC WORKS AND WATER RESOURCES
RESERVOIRS AND GRAND BARRAGES SECTOR

# Naga Hammadi Barrage Development

# FEASIBILITY STUDY
# FINAL REPORT

**AUGUST 1997**

VOLUME 1

MAIN REPORT

**NAGA HAMMADI BARRAGE DEVELOPMENT CONSULTANTS**
Consortium Lahmeyer Electrowatt Sogreah

In association with
Arab Consulting Engineers
Moharram - Bakhoum
Egypt

## Naga Hammadi Barrage Development

# FEASIBILITY STUDY
# FINAL REPORT

**VOLUME 1 : MAIN REPORT**

VOLUME 2 : APPENDICES

- A Terms of Reference for Phase 2 - Feasibility Study
- B The Existing Barrage Structures
- C The Existing Head Regulators
- D Topographic Documentation and Survey
- E Geotechnical Conditions and Investigations
- F Geotechnical Aspects of Construction Pit and Foundation
- G Construction Materials

VOLUME 3 : APPENDICES

- H Water Resources
- I Assessment of the Function of the Toshka Flood Release Canal
- J Emergency Release and Design Flood Propagation through Lake Nasser
- K Hydraulics of the River Reaches Upstream and Downstream of NHB

VOLUME 4 : APPENDICES

- L1 Flood Release Concept and Sluiceway Dimensions
- L2 River Diversion during Construction
- L3 Flow Conditions and Riverbed Protection
- L4 River Navigation and Design of Navigation Lock
- L5 Transient Flow Conditions
- L6 Specifications for Hydraulic Model Tests
- M Groundwater Modelling

VOLUME 5 : APPENDICES

- N The Unified Power System (UPS)
- O Mechanical Equipment of the Powerhouse
- P Electrical Equipment of Powerhouse and Transmission
- Q Estimation of Energy Generation and Selection of Installed Capacity
- R Naga Hammadi Mini Hydropower Plant
- S Hydraulic Steel Structures of the New Barrage
- T Construction Details and Cost
- U Contingency Assessment

VOLUME 6 : APPENDICES

- V Evaluation of Cross River Traffic
- W Economic Evaluation of Agricultural Production
- X Economic Analyses Supporting Documentation
- Y Financial Analyses Supporting Documentation
- Z References

VOLUME 7.1 : ENVIRONMENTAL IMPACT ASSESSMENT - MAIN REPORT

VOLUME 7.2 : ENVIRONMENTAL IMPACT ASSESSMENT - ANNEXES

VOLUME 8 : ALBUM OF DRAWINGS

# MAIN REPORT

## TABLE OF CONTENTS

## LIST OF TABLES

**LIST OF FIGURES**

**Institutions Frequently Mentioned:**

| | |
|---|---|
| MOPWWR | Ministry of Public Works and Water Resources |
| RGB | Reservoir and Grand Barrages Sector |
| GANT | General Authority of Nile Transport |
| MEE | Ministry of Electricity and Energy |
| HPPEA | Hydropower Projects Executing Authority |
| EEA | Egyptian Environmental Affairs Agency |
| KfW | Kreditanstalt für Wiederaufbau |
| NHBDC | Naga Hammadi Barrage Development Consultants |
| NRI | Nile Research Institute, Quanater |
| HRI | Hydraulic Research Institute, Quanater |
| GRI | Groundwater Research Institute, Quanater |
| PIU | Project Implementation Unit |

TABLE OF CONVERSION BETWEEN UNITS USED IN THE REPORT AND THOSE COMMONLY USED BY OTHERS

| Item | Conversion of Units | |
|---|---|---|
| Area | 1 feddan | 4,265 $m^2$ |
| Volume | 1 $hm^3$ | $10^6$ $m^3$ |
| Volume | mcf = million cubic feet | 28,317 $m^3$ |
| Volume | 1 bbl | 0.159 $m^3$ |
| Flow Rate | million $m^3$/day | 11.57 $m^3$/s |
| Flow Rate | billion $m^3$/month | 386 $m^3$/s |
| Flow Rate | billion $m^3$/year | 31.71 $m^3$/s |
| Calorific Value/Energy | 1 MBTU | 1.055 GJoule |
| Calorific Value | 1 MBTU | 0.252 Kcal |
| Calorific Value/Energy | 1 BTU | 1.055 KJoule |
| Calorific Value/Energy | 1 KJoule | 0.948 BTU |
| Calorific Value/Energy | 1 GJoule | 0.239 Mcal |
| Calorific Value | 1 Kcal | 3.968 BTU |
| Calorific Value/Energy | 1 Kcal | 4.186 KJoule |
| Weight | 1 t Crude Oil | 7.26 bbl Crude Oil |
| Weight | 1 t Mazoud | 6.64 bbl Mazoud |
| Currency | 1 US$ | 3.40 LE |
| $CO_2$ Emissions of Thermal Generation from Natural Gas | 56 t/TJoule | 0.42 t/MWh |

# 1. BACKGROUND AND STARTING POINT OF FEASIBILITY STUDY

## 1.1 INTRODUCTION

The present Report is the third stage in a successive refinement in the development of the Naga Hammadi Barrage from the commencement of the study through to the detail of full feasibility design. To date, a Conceptual Study was undertaken in 1993, followed by an Interim Study in 1994.

The original Terms of Reference for the Feasibility Study, which were amended by those of the Interim Study and Feasibility Study as shown in Appendix A, required that the development of Naga Hammadi Barrage includes a Hydropower station.

In 1996, a draft of the feasibility study was discussed between representatives of the Reservoir and Grand Barrages Sector (RGB) within the Ministry of Public Works and Water Resources (MOPWWR) and of the Hydropower Projects Executive Authority (HPPEA) and the Egyptian Electricity Authority EEA which resulted in amendments of the criteria to which the New Barrage and hydropower plant was to be designed. The most important of these criteria were that the headpond level shall be maintained at 65.9 m asl and the operation of the hydropower plant shall be restricted to run-of-river generation. The present Feasibility Study takes fully into account the amended criteria as outlined in Appendix A, Aide Memoire, 1996.

The alternatives initially conceived for the development of the Naga Hammadi Barrage were rehabilitation of the existing barrage, rehabilitation of the existing barrage with the addition of a hydropower station, and construction of a new barrage with a hydropower station. Constraints imposed by the existing structure and local geology in the immediate vicinity of the existing barrage preclude construction of a hydropower station there. Even rehabilitation of the existing barrage structure, dated from 1930, appeared to involve an unquantifiable risk regarding the reliability of the basic measures proposed. Although technically viable they could not guarantee the length or reliability of the extended service life. Thus, the only feasible option to develop the hydropower potential at Naga Hammadi is to include a power station in a new barrage located at a site offering geological conditions suitable for establishing and temporarily maintaining a large construction pit on the River Nile.

In order to quantify the merits of including hydropower in a new barrage, as required by the Terms of Reference, it was necessary to study two options to a comparable level of detail:

- layouts entailing a new barrage with hydropower referred to as the 'New Barrage', and
- layouts entailing a new barrage without hydropower referred to as the 'New Barrage without Hydropower' (Base Case).

The studies leading to this Feasibility Report were also accompanied by five meetings with a Panel of Experts (POE) comprising internationally recognised experts providing technical advise to the MOPWWR.

## 1.2 PROJECT ENVIRONMENT

The Naga Hammadi Barrage (NHB), referred to in the Report as the 'Barrage', was constructed on the River Nile from 1927-1930. It is located some 12 km to the north of the town of Naga Hammadi. The location in the context of both Upper Egypt and the more immediate environs is shown on Album No. 1. The location of the Barrage, generally expressed as a river distance in kilometers downstream from the Aswan Dam, is at km 359.50.

The Barrage is one of three structures on the River Nile in Upper Egypt (excluding the Aswan and High Aswan Dams) which control the water levels for some distance upstream. The others include Esna Barrage located to the south at km 166.65 and Assiut Barrage to the north at km 544.75. The river reaches between Esna and Naga Hammadi Barrages and Naga Hammadi and Assiut Barrages are 192.85 km and 185.25 km respectively.

The primary purpose of the Naga Hammadi Barrage, as also for the other two barrages, was to expand the cultivated areas in Upper Egypt by raising the water levels to enable irrigation all year around rather than only during periods of flood. For this purpose, head regulators for the main two irrigation canals, Naga Hammadi El-Sharkia on the east side and Naga Hammadi El-Gharbia on the west side of the river, were constructed in 1930, at approximately the same time as the Barrage. In addition, the increased water levels are required to meet navigational requirements. To a lesser extent, domestic and industrial water supplies also benefit.

The High Aswan Dam (HAD) was constructed in the late 1960's to regulate the flow of the River Nile in Egypt. Prior to its construction, the country was subjected not only to large seasonal variations in river discharges (the flood months being August to October) but also to sequences of years with below-average flow. Since the implementation of the High Aswan Dam with its active storage capacity of $90x10^9m^3$ and flood retention storage of $41x10^9m^3$, the River Nile discharges have been fully regulated. Maximum discharges at Naga Hammadi during the peak irrigation period from June to August are now around 2,400 $m^3/s$ and reduce to less than 700 $m^3/s$ during the winter closure period of the irrigation systems for a three to four week period from mid-December to mid-January. Far lower minimums of 350 $m^3/s$ actually occur within this periods for a number of days. After 1968, floods as previously known, have not been experienced. Hence, the operation of the Naga Hammadi Barrage has been substantially changed and the full capacity of the 100 vents for release of floods (maximum capacity of about 14,000 $m^3/s$) are no longer required.

The reduction of floods by the HAD has resulted in the stabilizing of the river course laterally and a reduction in the width of the riverbed. However, a process of dynamic change has been observed to varying degrees along the river course associated with riverbed degradation.

The Nile Valley north of Aswan follows a northerly to northwesterly direction. Between Luxor and Naga Hammadi, the river bends substantially before again continuing in the original direction. This is considered to reflect the influence of major faults and geologic structural controls in the region (MOPWWR, 1992). This is particularly the case for the eastward-trending reach of the river, which also parallels the trend of tributaries in Wadi Qena (directly north of Qena). The slope of the valley floor between Aswan to a point midway between Naga Hammadi and Assiut is of the order of 0.075 m/km.

The floodplains adjacent to the river are utilized almost entirely for agricultural production based on irrigation. Precipitation is negligible (averaging less than 1 mm annually), resulting in no runoff from surrounding areas apart from the very occasional occurrence of localised floods from intense but extremely short rainfalls.

The reach between Naga Hammadi and Esna Barrages falls within two Governorates, those of Sohag and Qena. At the Barrage, the west bank and the El Dom island are within the Governorate of Qena, while the east bank is within Sohag (see border on Album No. 1). In the vicinity and upstream of the NHB a number of population and industrial centres exist in addition to numerous local villages. Many rely on the River Nile to satisfy current water demands, although groundwater from the underlying aquifer system supplies villages located some distance from the river. The locations of major towns in the region are indicated on Album No. 1.

The regional population is in the vicinity of 2.5 million and, based on the average annual growth rate observed in Egypt, is increasing at around 2.3% per annum. The economic livelihood of the majority

of the population is based on agricultural production, although tourism and several large industrial factories also play a role in the local economy. Upstream of the Barrage, a number of larger cities have been established. These include Naga Hammadi, Deshna, Qena and Luxor, located respectively at distances of 12, 42, 73 and 136 km. Apart from providing services for the local agricultural economy, these and other smaller cities have also seen the establishment of relatively large industries, including the Aluminium City complex near Naga Hammadi and the sugar factories at Naga Hammadi, Deshna and Kous. The city of Luxor has developed as the major tourist centre in Upper Egypt.

## 1.3 PURPOSE OF THE BARRAGE

The purpose of the Barrage is to maintain the water level in the headpond at an appropriate elevation primarily for the diversion of irrigation water to the two major side canals, the Western and Eastern Naga Hammadi canals. These branch off at river km 358.5 and 359.2 upstream of the Barrage, extending for some 211 and 157 km into the downstream irrigation area. The Western NH canal currently supplies a total area of 439,000 feddan of irrigated land, and the Eastern NH canal a total of 119,000 feddan.

New land is currently being reclaimed from desert areas downstream of the Barrage. Provided the headpond level is maintained at 65.9 m asl, some 0.5 m above the current maximum level during summer, an additional 40,000 feddan can be supplied from the Western NH canal and 31,000 feddan from the Eastern NH canal directly from the headpond. A level of 65.9 m asl, regarded to be the former design level of the Barrage is considered as the level to which also the New Barrage shall be designed. In addition to the diversion of river discharge to the side canals, there are agricultural areas upstream totaling 122,774 feddan which depend on pumping of water from the river by 6 large pumping stations.

Overall, the total areas to be supplied from the Naga Hammadi headpond will increase to around 751,800 feddan (3,206 $km^2$) in the near future. The annual volume of water abstracted to meet the requirements of these areas is around $5x10^9m^3$. A summary of the agricultural production for areas which depend on the continued supply of irrigation water from Naga Hammadi Barrage, including the value of their annual production as derived in Appendix W, is presented in Table 1.1.

Table 1.1: VALUE OF AVERAGE ANNUAL AGRICULTURAL PRODUCTION FOR IRRIGATION AREAS SERVED BY NAGA HAMMADI BARRAGE

| Location/Supply | Area $10^3$feddan | Annual Production $10^6$LE |
|---|---|---|
| Old Land: | | |
| Western Naga Hammadi Canal | 439.0 | 1,030 |
| Eastern Naga Hammadi Canal | 119.0 | 280 |
| Pumping Stations: | | |
| Abu Homar | 16.5 | 31 |
| El Khayam | 43.3 | 81 |
| Derb | 27.7 | 52 |
| Hamoudeya | 0.3 | 0.6 |
| El Marashda | 35.0 | 65 |
| New Land Under Development: | | |
| Western Naga Hammadi Canal | 40.0 | 94 |
| Eastern Naga Hammadi Canal | 31.0 | 73 |
| Total | 751.8 | 1,707 |

As mentioned, an additional function of the Barrage is to facilitate navigation along the river reach between the Barrage and Luxor. As a result, it was constructed with a navigation lock with a usable chamber length of 70 m and width of 16 m. Due to river degradation following implementation of the HAD, the minimum water level above the end sill of the original lock chamber was reduced to the extent that river traffic, and particularly barges and large passenger ships, could not pass through during a considerable period of the year. As a result, a new navigation lock was built to the west of the existing lock (see Album No. 56) with the same overall dimensions of the lock incorporated at the upstream New Esna Barrage. Inaugurated in 1994, this also incorporated a sufficiently low end sill to account for further river bed degradation.

In view of the large regulating effect of the HAD, the generation of energy will be generally constant from year to year, and generation over the year would follow the schedule of releases from the HAD made for the purpose of irrigation. Hence, the operation of the hydropower plant will not have any effect on the regulation of river flow.

By instruction of the MOPWWR, the design headpond level of the existing Barrage, which would also be the level of the New Barrage with hydropower, will be 65.9 m asl. This will remain generally constant over the year.

Present operating water levels (since 1994) deviate from this level by a maximum of some 0.8 m, with average summer and winter water levels being constant at around 65.4 m asl and 65.1 m asl respectively. Maintaining a headpond lower than the design in summer limits the water abstractions to the main irrigation canals. The lower headpond level in winter, however, assists in improving the factor of safety for the stability of the downstream apron of the existing Barrage. Re-establishment of the design headpond level with the New Barrage would therefore result in impacts such as increased groundwater levels in irrigated areas upstream, with associated effects on sewage, and so on. These would require investment for their mitigation, the extent of which can only be accurately assessed using groundwater modelling to quantify the impact over the Project Area. Conversely, positive effects will also result from a higher headpond level, such as reduced or even avoided pumping to irrigated land or land to be developed in the future.

As the Barrage with Hydropower would be developed as a new project downstream of the existing structure, the water level rise in the intervening river reach (by some meters) is considerable. The resulting impacts on the adjacent land area would therefore also be significant after impoundment.

## 1.4 REVIEW OF LAYOUT STUDIES FOR BARRAGE DEVELOPMENT

### 1.4.1 Assessment of the Present Conditions of the Barrage

The Barrage has functioned for 66 years as an irrigation water supply for a very large agricultural area which is of significant benefit and strategic importance to the Nation. The basis for all further studies of the New Barrage development must therefore be considered as an assessment of the present structural conditions of the existing Barrage and its level of reliability to guarantee a long-term continuation of irrigation supply.

As a result, an extensive investigation of the conditions of the existing Barrage was undertaken at Conceptual Study level. This included detailed inspections of the civil works and hydraulic steel structures, and exploratory drilling in the piers and foundation concrete to a maximum of some 36 m depth below the foundation level in sand and gravel layers. In addition, a thorough analysis on the stability of the structure under critical load conditions was carried out. Details are presented in Appendix B of the Feasibility Report. The result was that the structural integrity of the Barrage is at present still acceptable. However, in the near future a programme of complete rehabilitation of the Barrage and its downstream apron would be essential to meet the basic requirement of the Barrage, that is to ensure the continued supply of irrigation water to the downstream areas. A programme of

rehabilitation works was then developed within the Interim Study. This highlighted a number of critical factors related to:

- the success of the compaction grouting of the foundation soil,

- the technical difficulties of grouting works between the piers and the uncertainties of being able to extend the grouting under the apron slab.

- the relatively high apron slab and adjacent new rip-rap counterweir on which energy dissipation would take place during low tailwater levels.

- the rehabilitation of the 66-year-old gates, which appears to be possible now. However, if this would be the case in another 40 years (30 years after 2005) cannot be predicted with any certainty.

In discussions with the POE following the presentation of the assessment of the Barrage conditions and a possible rehabilitation program, the panel concurred that Barrage rehabilitation appears to be technically viable but would involve considerable risks. Whilst a reasonable number of geotechnical and construction material investigations were carried out overall, it can never be determined with certainty that these encompass the full range of site conditions. Hence, there remains a risk in the reliability of some of the basic measures of rehabilitation resulting in an uncertain remaining lifetime. In other words, although the rehabilitation measures are technically viable, it is not possible to guarantee the function of the Barrage for a reasonably long period which is ultimately the basis on which it would be judged an acceptable option. The rehabilitation option of the existing Barrage was therefore discarded from further consideration.

### 1.4.2 New Barrage with Hydropower

During the Conceptual Study, an appropriate layout of a New Barrage with hydropower was identified with the axis on the downstream river bend of the River Nile around Geziret el Dom, at river km 362.49. Studies of the layout at that time could not be supported by geotechnical investigations, and hence were inconclusive regarding the technical viability of this layout. Therefore, it was decided to continue with geotechnical investigations in the larger area of the New Barrage layout.

In the course of the engineering evaluation, different options for the construction pit and diversion scheme were investigated:

(i) Construction pit for all concrete structures located within the riverbed, diverting the river flow through a side canal around the construction pit. The diversion canal would be either on the left bank of the river or to the right on El Dom Island.

(ii) Construction pit on El Dom Island. The existing river channel to the left of the island would remain unaffected as a result of construction. Following completion of the concrete structures of the New Barrage, the river channel would be relocated through the New Barrage. The original river bend to the left of El Dom Island would be permanently closed by an embankment dam.

Geotechnical investigations, including drilling and soil sampling, undertaken at both alternative locations for the New Barrage revealed a continuous clay layer at about 35 to 45 m depth below present river levels exists only in the area of option(i) below the river bend and adjacent left (western) bank, but not at site (ii) on the island.

The decision was then taken to adopt the site in the original river course for the New Barrage. This was supported by:

- The existence of the clay layer to which seepage-cutoff walls of the construction pit could be connected, which contributes to reduced risks during construction and significantly reduces costs of pumping installations and their operation.

- The minimal disturbance to the environment during construction and the additional area of reclaimed land in the existing flood channel to the right of El Dom Island suitable for perennial cropping, resulting in a positive balance in the type of land after construction; and

- The significantly smaller quantities of earthworks involved in the construction.

The selected layout of the New Barrage was then subject to further geotechnical investigations and refinements in design with the aim of reducing the overall quantities, extent of the construction period, and costs.

### 1.4.3 New Barrage Without Hydropower

For the purpose of the economic evaluation of hydropower (Chapter 9), the hydropower costs are separated by deducting the cost of a New Barrage without Hydropower from that of a New Barrage.

The headpond level of the New Barrage without Hydropower would be the same as for the "with hydropower" case, as the MOPWWR confirmed that the design headpond level of the existing Barrage should be re-established for any new barrage structure. The design level will also provide significant improvements in control of water abstractions through the head regulators of the irrigation canals branching off the headpond following their rehabilitation. Furthermore, the original design headpond level would allow the supply of water directly from the headpond to the new downstream irrigation areas (see Album No 84) now under development. This would allow irrigation of these areas without pumping from the river downstream of NHB.

Hence, for the sole purpose of economic evaluation, the New Barrage without Hydropower was developed with a headpond level of 65.9 m asl and construction quantities, costs, and construction schedule established.

The requirements for the depth of the construction pit for the sluiceway and navigation lock are less severe than for the case with the hydropower plant, and the area of the construction pit is smaller. The location and shape of the diversion canal remains unchanged compared to the "with hydropower" case. In order to achieve similar operating conditions during the passage of floods as those of the New Barrage, seven sluiceway openings of the same dimensions were adopted as for the "with hydrpower" case, limiting the reservoir surcharge to a level which complies with the allowable headpond level rise during passage of the 'emergency flood'.

The remainder of the layout is based on the same criteria as applied in the layout of the New Barrage.

## 1.5 ENVIRONMENTAL EFFECTS

The present headpond levels of the existing Barrage, namely 65.1 m asl in winter and 65.4 m asl in summer, were assumed to be representative of the actual environmental conditions associated with the river and groundwater levels upstream of the Barrage. The increase from these levels to a constant level of 65.9 m asl will result in increased river levels upstream. This will result in localised impacts on the riverbanks and river islands and higher groundwater levels in parts of the river plain

area. Therefore, as the basis for an accurate assessment of environmental affects of the New Barrage, it was necessary to:

- Undertake field surveys to collect physical and socio-economic data to provide an overview of existing baseline conditions against which the severity of project-related impacts could be assessed.

- Assess construction-related impacts on land use and local communities in the near vicinity of the construction site (primarily Dom Island and left bank).

- Establish a more accurate goundwater model extending over the Project Study Area (some 65 km upstream and 20 km downstream of the New Barrage).

- Install some 53 new observation piezometers in the Project Study Area and monitor the groundwater levels for an extended period to provide basic data suitable for the calibration of the groundwater model and quantify ongoing changes in the groundwater levels in response to irrigation and seasonal changes in river flows and levels.

The magnitude of the impact resulting from re-establishing the design headpond level of the Barrage, and its subsequent effect on groundwater levels, is a function of the adequacy of the present drainage network in the area. Inspections of large sections of the Project Area indicated the main open drain system is not being properly maintained at present with excessive growth of weeds and water hyacinth in the main open drains. This is considered to be a major contributing factor to the existing high groundwater levels observed over much of the area even with the present lower headpond levels. Under existing conditions of the drainage network, re-establishing the former design headpond level of the Barrage would further compound these effects. The incremental effects would, however, be significantly larger than if the open drain network were properly maintained. Hence, for the purpose of identifying the project impact, groundwater simulations were undertaken assuming as a starting point the implementation of a sustainable maintenance programme of the main open and secondary drains in selected sections of the Project Area. It must be ensured that this programme is carried out as part of the overall maintenance of the drainage network by the Drainage Directorate.

## 1.6 REHABILITATION OF HEAD REGULATORS

The rehabilitation of the head regulators for the Western and Eastern Naga Hammadi canals is a firm requirement for a New Barrage with or without hydropower. This is necessary simply to achieve the normal level of safety against uplift and sliding as described in Chapter 4 and Appendix C. The relevant rehabilitation measures, including automization of gates, are also described. The cost estimates for the rehabilitation are consequently included for both New Barrage cases.

## 1.7 LONG TERM SCENARIO OF DISCHARGE AND HEAD CONDITIONS

The releases from HAD are determined primarily in accordance with downstream irrigation demands. Since commissioning of the HAD there has been a change in the magnitude of the seasonal releases reflecting both changes in the size of the agricultural areas supplied and also variations in cropping patterns. The total annual volume of releases has, however, remained essential constant and in line with the volume of release Egypt is permitted under its Treaty with the Sudan for operation of the HAD.

The flow distribution now observed (since 1994) exhibits larger discharges during the summer period with commensurate reductions during the winter period than observed through the 1970s and 1980s. The flows during the period of winter closure are also now substantially lower than through those years. On this basis (and as discussed in Chapter 6 and Appendix H) the flows for the last two years were adopted as representative of future conditions for the estimation of energy generation, as also

agreed with the MOPWWR. It can not, however, be disregarded that further redistributions of the flow may occur into the future as more extension areas are developed for agriculture and water savings become possible through the implementation of new cropping regimes, reduced water losses from the irrigation systems, and so on.

Two possible scenarios were considered to asses the impact on future discharge and head difference scenarios at the New Barrage and their impact on energy generation. These included the development of the proposed Toshka pumping station to develop extensive agricultural areas in the 'New Valleys'. This will result in the abstraction of water directly from Lake Nasser (equal to around 15% of the current annual release from HAD when the scheme is fully developed) and a commensurate reduction in River Nile flows downstream. Also river degradation, which was substantial immediately after commissioning of the HAD, is continuing although at a much reduced rate (Appendix K). Estimates indicate the possible long-term degradation will be some 0.8 m (equal to around 15% of the rated net head at the New Barrage).

Both are more extreme scenarios, that is representing maximum long-term degradation or significant abstractions of water from Lake Nasser. They will result in opposite effects in terms of energy generation. However, when considered either individually or combined they have relatively little impact on the total annual energy estimates. Hence any possible future developments are unlikely to significantly affect the performance of the New Barrage in this regard.

## 1.8 UTILIZATION OF THE BARRAGE FOR PUBLIC TRAFFIC

The original Terms of reference of the Study from 1993 required that a public road bridge on the barrage spanning the River Nile shall be incorporated in the design. Subsequent discussions with the MOPWWR and the POE on the requirement for a high level road and related cost indicated the need for:

- Design of the New Barrage with 3 bridge alternatives of substantially different cost, namely a service bridge (not permitting public traffic), a low level road bridge, and a high level road bridge. The latter avoids any interruption of road traffic when sluicing large vessels through the navigation lock.

- An assessment of the present and future traffic volume for justification of a public road bridge and its level on the barrage.

## 1.9 OLD MINI HYDROPOWER PLANT AT NAGA HAMMADI

The old mini hydropower plant was established in 1942, taken out of operation in 1971, and is presently being rehabilitated. It will be recommissioned in 1997. In view of the intended implementation of the New Barrage with hydropower component, rehabilitation works were minimized and mainly involve the electrical and some parts of the mechanical equipment. The financial evaluations indicated that the amounts spent for rehabilitation would have been amortized at the time when the New Barrage starts operation, however, the service life of the rehabilitated plant would be 10 to 20 years longer.

The mini hydropower plant would cease operation in both cases - New Barrage with or without hydropower. Therefore, although it would not affect the economy of hydropower generation of the New Barrage, a preliminary evaluation of the possibility of transferring the equipment to the Zifta Barrage in the Delta was carried out. This is described in Appendix R. With this transfer, the investment made in the refurbishment at the minihydropower plant at Naga Hammadi would be saved with continued generation at Zifta.

## 1.10 ALLOCATION OF PROJECT COMPONENTS TO THE CASES OF ECONOMIC AND FINANCIAL EVALUATIONS

The schemes described above form part of two individual projects, namely the New Barrage and the New Barrage without Hydropower. The New Barrage without Hydropower is considered as the so-called Base Case for economic and financial evaluation for the purpose of separating the hydropower cost from the total project cost. For both projects, the extent of the environmental impacts (side effects) is almost the same.

Whereas the above cost separation of the hydropower cost must be used for the economic evaluation, the financial evaluation mainly considered a cost split agreed between the MOPWWR and the MEE. According to this agreement, the Electricity Sector (EEA) only assumes the costs of electrical and mechanical equipment financially involved in the hydropower station.

Both economic and financial evaluations are only carried out for the hydropower plant. In view of the estimates of agricultural production based on irrigation supplies from the headpond of the Naga Hammadi Barrage given in Table 1.1, the annual income from irrigation of the areas supplied is in the order of 1.5 times the cost of the New Barrage without Hydropower. Hence, it is concluded that guaranteeing continued irrigation by the project is a necessity which does not require separate economic evaluation.

The financial analysis is only meaningful in relation to revenues by which the debt service of the project can be paid. As the MOPWWR does not dispose of direct revenues from irrigation, the financial analysis is also restricted to the hydropower component.

## 1.11 STARTING POINT FOR THE FEASIBILITY LEVEL STUDIES

Following the conclusion of the Conceptual and Interim Phases, which were regarded as preparatory stages of the Feasibility Study, the Terms of Reference were reviewed and adapted to the situation prevailing at the end of the Interim Study. These are presented in Appendix A. Subsequent to submission of a draft of the Feasibility Study, results were intensively discussed between the RGB/MOPWWR, with the Panel of Experts, and with the MEE represented by the HPPEA and EEA. These discussions lead to an amendment of the Terms of Reference based on the Aide Memoire, also contained in Appendix A.

As required by the amended terms, the Feasibility Study was carried out for a headpond level of 65.9 m asl for the New Barrage. The Study thereby concentrated on a continuation of research and field work, design, cost estimation, and economic and financial evaluations at full feasibility level.

The main research and field work concerned physical conditions and comprise the following areas:

i The design floods for which the sluiceway and other structures participating in flood evacuation are dimensioned, namely:

- the assessment of flood releases from the HAD for flood inflows with a 1:10,000 year recurrence interval, and
- the assessment of the emergency release from the HAD, to achieve limited drawdown of Lake Nasser;

ii. Exploration of the extent, depth, orientation and thickness of the clay layer in the area of the construction pit;

iii. Assessment of in situ soil permeabilities based on a pumping test at the future construction site;

iv. Groundwater modelling in upstream areas assisted by installation of piezometers and subsequent recording;

v. Detailed environmental impact analysis with surveys of present conditions in the construction and upstream areas, and development of an Environmental Management Plan.

The starting point for the design work, in conjunction with supporting hydraulic calculations, was the layout of the New Barrage selected from the Interim Study.

With the aim of minimising interference to navigation resulting from the approach flow to the hydropower plant, the sluiceway is located between the powerhouse and the navigation lock. With the further aim to minimise impacts on river morphology, the powerhouse is located on the left river side. Therefore, the navigation lock is on the right bank.

In the discussions with the POE, it was concluded that this configuration of the main project components will be applied to develop the final layout. The hydraulic behavior of the final layout would, after completion of the Feasibility Study, be subject to hydraulic model testing. Only if the hydraulic behavior did not satisfy expected criteria, would this configuration be modified. It was however requested that, a layout with the navigation lock on the left bank shall be included in the Album of Drawings (see section 4).

# 2. PHYSICAL CONDITIONS AND ASSESSMENT OF DESIGN PARAMETERS

The work described in this Chapter summarizes the engineering analyses and data collection undertaken in the three phases of the Feasibility Study for the purpose of designing the New Barrage layout and assessing the effect of a headpond level of 65.9 m asl. Additional studies associated with the environmental impact assessment are dealt with separately in Chapter 7.

## 2.1 AVAILABLE MAPPING, TOPOGRAPHIC AND BATHYMETRIC SURVEYS

During the study various topographic and hydrographic mapping were obtained for use in the engineering analyses. In addition a number of bathymetric surveys were undertaken in the vicinity of the Barrage.

*Topographic Mapping*

Topographic mapping was available for the Project Study Area at scales of 1:10,000, 1:25,000, and 1:50,000. Mapping at 1:10,000 scale, with 0.5 m contour intervals, was produced in 1978 from aerial photography by Kenting Earth Sciences (at the same scale). This extends the entire length of the Project Area but only provides a limited contour coverage some 1 to 2 km either side of the river channel. This mapping, obtained from HADSERI, has yet to be officially approved by the Egyptian General Survey Authority (EGSA).

Additional 1:10,000 scale mapping with 0.25 m contour information was also obtained from the Drainage Directorate. This was based on terrestrial surveys in 1984 during the installation and replacement of the tile drainage networks which cover much of the project area. It extends over approximately 80,000 feddan (340 $km^2$) and only excludes sections of the project area where tile drains are not yet installed, namely to the east of Naga Hammadi town and the Salam drain area to the southwest of Abu Tesht.

A complete coverage of the entire project area was provided at 1:25,000 scale with 0.5 m contour intervals. These maps are, however, significantly older and based on survey information produced in 1937. Their accuracy is considered less reliable.

Mapping at 1:50,000 scale based on 1988 aerial photography is currently under preparation. Although only of limited use regarding contour information, it did provide some details of the more recent infrastructural developments in the wider project area relevant to the groundwater modelling and environmental assessment.

The piezometric boreholes installed during the study were surveyed based on the available benchmarks in the area. It was not possible to obtain details on the new GPS stations from the Egyptian Survey Authority to match the measured coordinates to the national grid. Therefore adjustments were made based on the El Zajraa control, Cairo to provide overall consistency with other project-related surveys and mapping.

*Hydrographic Mapping*

Hydrographic mapping of the river channel was available at 1:2,000 and 1:5,000 scales, both with contour intervals of 0.5 m. These were based on the same hydrographic survey by Kenting Earth Sciences in 1981-82. The maps obtained at the larger scale extended approximately 5 km up- and downstream of the Barrage. Those at the smaller scale extended a considerably longer distance in both directions.

*Bathymetric Data*

A significant amount of cross-sectional data defining the channel of the River Nile were available for the reaches both up- and downstream of the Barrage based on field surveys undertaken periodically since 1963. The most important of the earlier surveys was in 1981/1982 by Kenting Earth Sciences (1983) in which river cross-sections were surveyed at intervals of approximately 5 km.

To supplement these data, three hydrographic missions were undertaken by the Nile Research Institute (NRI) as follows:

Conceptual Study:
- (i) Upstream reach from km 280 to km 359 at intervals of 2 km
- (ii) Downstream reach - km 360 to km 383 at intervals of 0.5 km
  - km 383 to km 398 at intervals of 1 km
- (iii) Vicinity of New barrage from km 360.5 to km 366.0 at intervals of 0.5 km over the first 22 km and 1 km over the next 18 km.

These served three purposes, firstly to provide a comparison with the earlier survey work to indicate any more recent changes in river geometry. Secondly it allowed a more quantitative judgement of the use of the earlier survey information to augment the recent data. Thirdly it provided far more detailed bathymetric information of the reach immediately up and downstream of the axis of the New Barrage required to undertake the two-dimensional flow field modelling and provide a basis for the future hydraulic modelling.

Overall, the shape of the cross-sections surveyed in both the up- and downstream reaches during the Conceptual Study showed good agreement with the 1982 survey data. On this basis it was possible to augment the data from the second survey of this study (downstream of the Barrage) with information from the 1991 survey (undertaken at 1 km intervals). This provided a comprehensive set of river cross-sections extending from Naga Hammadi to Assiut Barrages based on the most recently available surveys.

*Formulation of Mapping for Project Site*

The unavailability of both topographic and hydrographic mapping at a single, suitable scale for use in the project, required that a composite map of the immediate project area be developed. This involved a number of stages. Firstly, the topographic mapping at 1:10,000 scale (Kenting, 1978) was modified to 1:2,000 and combined with the hydrographic mapping of the river channel, also at a scale of 1:2,000 (Kenting Hydrographic Survey, 1981). The accuracy of the hydrographic contours from this mapping had been verified and augmented from the initial hydrographic survey during the Conceptual Phase, albeit with sections at approximately 500 m intervals. Finally, the results of the recent more detailed hydrographic survey in the immediate vicinity of the Barrage were used to further review the accuracy of the river channel contours and immediately adjacent riverbank areas. Any changes considered necessary were incorporated.

## 2.2 WATER RESOURCES MANAGEMENT AND ADOPTED STREAMFLOWS AND HEADPOND LEVEL

The River Nile below the HAD is the principal source of water supply within Egypt. Negligible natural inflow occurs downstream of HAD, with the discharge in the reaches between the dam and various barrages being determined totally as a function of the actual releases from the reservoir, abstractions for irrigation and other water demands, and return drainage flows. Since its commissioning, despite a relative extreme sequence of low inflows through the mid-1980s there have been no periods when the reservoir has failed to meet the required demands. In addition, larger inflows which result in the reservoir increasing above its normal operating level are now discharged through the Toshka spillway (as discussed below). Therefore the historic annual releases from the

HAD and their associated patterns can be considered as representative of the outflow sequences which can be expected over the long-term with some qualifications as discussed below.

### 2.2.1 Operation of High Aswan Dam

The High Aswan Dam (HAD) was constructed in the late 1960s to regulate the flow of the River Nile in Egypt. Located some 360 km upstream of Naga Hammadi Barrage, the reservoir has a capacity at full supply level of some 137.5 $10^9 m^3$ with an active storage of 100.3 $10^9 m^3$. This can be compared to the longterm yearly average inflow of 84 * $10^9 m^3$. The present operation of the HAD is governed by a 1959 Treaty, 'Agreement for the Full Utilization of the Nile Waters', between the Governments of Egypt and the Sudan. The maximum annual release to Egypt under this Treaty is currently restricted to 55.5 $* 10^9 m^3$.

Although the primary purpose for the construction of the HAD from Egypt's perspective was the provision of a reliable water supply for agricultural production, the storage also serves to meet additional water requirements arising from municipal and industrial water demands, navigational needs, and importantly the generation of electrical energy. The latter is, however, perceived to be a consequence of releases to satisfy irrigation demands and not the primary criteria for the release. The operational policy for the reservoir is determined on an annual basis by the MOPWWR taking into account existing storage levels, projected demands, and historical release patterns.

In addition to providing flow regulation, the HAD also has significant impact on peak flood discharges now occurring downstream. Prior to its construction, river discharges generally reached a maximum around September averaging of the order of 8,700 $m^3/s$, although maxima around 10,000 $m^3/s$ were not uncommon. These peak flows occurred as a seasonal 'flood' which persisted at or near the maximum level for several months. This is a direct function of the huge contributing catchment area and very low gradients within the middle and lower reaches of the River Nile in the Sudan and Upper Egypt.

Post-HAD, the controlled releases now peak annually at around 2,300 $m^3/s$ at the Barrage (2,500 $m^3/s$ in its upstream reach) to match maximum irrigation demands from June to August and reduce to a minimum of approximately 900 to 1,000 $m^3/s$ during the winter period (excluding the period of winter release when discharges are curtailed for the purpose of maintenance on the irrigation system - referred to as `winter closure').

With an installed capacity of 2,100 MW at HAD, releases are generally discharged through the turbines. Although the volume of daily outflows are determined by irrigation requirements, the actual release pattern is regulated diurnally to generate power on a peaking basis, more efficiently satisfying the energy and power requirements of the Unified Power System (UPS) and particularly those of Southern Egypt. To ensure discharge fluctuations do not propagate downstream, the Aswan Dam located some 7 km to the north, provides reregulation. The original structure at Aswan Dam was constructed around 1900 but has undergone several modifications and upgrades. With an installed capacity of 571 MW, it is also used for hydropower generation.

### 2.2.2 Projected Streamflows

Hydrological data are available for a number of streamgauging stations located on the River Nile at which water level, stream discharge, sediment and water quality data are recorded. The collection of these data is the responsibility of the Irrigation Department within the MOPWWR.

Of particular relevance to this Study were the discharge and water level data for the existing Naga Hammadi Barrage, the latter being recorded both immediately up- and downstream. An analysis of the flows for the period since commissioning of the HAD (late 1960s) indicated that although the average annual flow at Naga Hammadi has varied only marginally (apart from a 3 to 4 year period

during the early 1980s), the seasonal distribution of flows since the early 1990s now differs significantly from the earlier period. There has been an increase in summer discharges (June to August) with average monthly flows during this period now of the order of 2 170 $m^3/s$. Daily discharges also now peak at around 2 400 $m^3/s$. In winter, the average flow from October to February is of the order of 890 $m^3/s$ but reduces to as low as 350 to 400 $m^3/s$ for short periods in December and January. This period, referred to as winter closure, marks the period when releases are reduced significantly from the HAD to allow maintenance of the irrigation systems throughout Egypt. This is undertaken in mid- to late-December in Upper Egypt and early to mid-January in Lower Egypt.

It was important that the flow sequence used for the simulation of energy generation be representative of the future release strategy implemented by the MOPWWR, both in terms of seasonal distribution and also magnitude of flows. Following discussions with the MOPWWR and based on a comparison of the current and earlier flows at Naga Hammadi Barrage, it was concluded that an average of only the last two years discharge from September, 1994 to August, 1996 be utilised. The average daily flow at Naga Hammadi Barrage over this period was 1,383 $m^3/s$ which is only some 3% lower than the long-term average based on the earlier period from 1968 (completion of the HAD) to 1994.

Finally, the MOPWWR indicated during discussions that future flows in the period of winter closure are likely to be reduced to a minimum of around 350 $m^3/s$ for an extended period and not only for 3 to 4 days as currently observed. This flow was therefore assumed for the second 10-day period of January, coinciding approximately with the period of winter closure in Lower Egypt. The 10-day flow sequence at the barrage and in the upstream reach for this period is shown graphically on Figure 2-1. The average monthly flows are presented in Table 2.1.

Also included are the maximum and minimum 10-day averages within each month. Based on the flows over the last two years, this would represent a 'saving' of around 0.15 $10^9 m^3$ or some 0.3% of the total annual release from the HAD. As it is likely that this 'saving' will be retained in Lake Nasr and released during future years when inflows to HAD are low, there was no attempt to redistribute this additional flow during the remainder of the year. The flow duration curve based on the adopted discharges is presented on Figure 2-2. The mean annual discharge for this flow series is 1 380 $m^3/s$.

Table 2.1: AVERAGE FLOWS AT NAGA HAMMADI BARRAGE

| Month | Average $m^3/s$ | Maximum 10-Day | Minimum 10-Day $m^3/s$ |
|---|---|---|---|
| January | 582 | 815 | 350 |
| February | 1,033 | 1,070 | 963 |
| March | 1,338 | 1,465 | 1,160 |
| April | 1,347 | 1,381 | 1,326 |
| May | 1,569 | 1,796 | 1,372 |
| June | 2,292 | 2,320 | 2,252 |
| July | 2,203 | 2,223 | 2,172 |
| August | 1,991 | 2,063 | 1,892 |
| September | 1,412 | 1,669 | 1,119 |
| October | 1,020 | 1,059 | 997 |
| November | 1,982 | 1,231 | 910 |
| December | 679 | 694 | 654 |
| Annual | 1,380 | - | - |

### 2.2.3 Projected Headpond Levels

Based on discussions between the MOPWWR, MEE, the KfW, and the Consultant prior to the commencement of the Feasibility Study, the decision was taken to adopt a headpond level of 65.9 m asl for the New Barrage. The MOPWWR indicated, however, that for some 10% of the year there could be reductions (of unspecified magnitude but within certain limits) to meet short-term increased water demands downstream of Naga Hammadi which could not be met in sufficient time by additional releases from the HAD. The headpond level of 65.9 m asl therefore formed the basis for the estimation of energy generation for which sequences of streamflow and headpond levels were required. During subsequent discussions with the MOPWWR at the commencement of the study, the requirement to incorporate these drawdowns in the analyses was confirmed.

A review of recent historic headpond level data suggests these drawdowns have to date occurred very infrequently and are only of the order of 0.05 to 0.10 m. The MOPWWR suggested future drawdowns could be significantly larger (varying from 0.1 to 0.5 m) and maintained over some 30 days annually. Their timing is, however, uncertain and it was therefore agreed that for the purposes of energy calculation, periodic drawdowns be introduced for three 10-day periods. The magnitude of the adopted drawdown was 0.25 m as an average for 10-day periods at the beginning of March and May and mid-October. The first period is approximately midway between winter closure and the high flow season, while the other two are at the beginning and end of the high summer demand period. (Subsequent analyses during energy calculations indicated the periods selected had almost no effect on energy generation).

The reduced generating head during these periods will result in a lower energy generation. However, the approach agreed with the Ministry is conservative for several reasons. Firstly, based on the recent historic operation of both Naga Hammadi Barrage and also the recently completed New Esna Barrage, the total period of the drawdowns is likely to be less than 30 days. Secondly, a review of the headpond level data for New Esna Barrage since its inauguration suggests the magnitude of the drawdown is also likely to be smaller than 0.25 m and probably of the order of 0.10 m.

### 2.3.4 Design Flows for Water Surface Profiles

Immediately upstream of the existing Barrage major irrigation offtakes including the Eastern and Western Naga Hammadi canals and smaller irrigation pumping stations abstract significant volumes of water from the River Nile. The abstractions vary seasonally with the summer and winter discharges from the river being of the order of 200 $m^3/s$ and 140 $m^3/s$ respectively. It was therefore recognised that the flows at the Barrage (see Section 2.2.2) were not representative of those in the upstream reach required for estimation of water surface profiles. The impacts of the abstractions and drainage returns in the river reach upstream for the concurrent two year period, September, 1994 to August, 1996, adopted for estimation of the flow sequence at the Barrage were therefore reviewed. This involved undertaking a water balance of the river reach from Naga Hammadi upstream to Esna. For the two year period the average abstractions over this reach were summed and added to the observed flow at Naga Hammadi. The accumulated flow compared well with the observed average discharge immediately downstream of the New Esna Barrage over the concurrent period.

Estimates of seasonal flows in the upstream reach were then derived using the same approach. A summary of the design discharges subsequently applied in the backwater analyses and the purpose for which they were derived is presented in Table 2.2.

Table 2.2: ADOPTED DESIGN FLOWS UP- AND DOWNSTREAM OF NAGA HAMMADI BARRAGE FOR WATER SURFACE PROFILE ANALYSES

| Design Discharge $m^3/s$ | Purpose | Method of Estimation |
|---|---|---|
| 2, 500 | - Determine need for protective measures at existing irrigation stations<br>- Determine areas of river islands available for perennial cropping | Maximum discharge upstream of Naga Hammadi Barrage during summer period |
| 2, 370 | - Determine need to install drainage pumping stations at existing outlets<br>- Groundwater modelling upstream of barrage for summer period<br>- Determine pumping energy requirements for irrigation and drainage pumping stations during summer | Discharge based on average flow upstream of barrage over months of June to August |
| 2, 170 | - Groundwater modelling downstream of barrage for summer period | Discharge based on average flow at barrage over months of June to August |
| 1, 600 | - Determine areas of river islands available for winter cropping | Maximum discharge upstream of Naga Hammadi Barrage during winter period |
| 1, 340 | - Groundwater modelling upstream of barrage for winter period<br>- Determine pumping energy requirements for irrigation and drainage pumping stations during winter (exclude 1 month for winter closure) | Discharge based on average flow upstream of barrage over months of November and February to April |
| 1, 200 | - Groundwater modelling downstream of barrage for winter period | Discharge based on average flow at barrage over months of November and February to April |

## 2.3 DESIGN FLOOD DISCHARGES

The regulation of the River Nile is now such that the downstream releases under all but the most extreme inflow conditions to the HAD are totally controlled by the adopted release strategy from the reservoir. This strategy can be based on a number of facilities including power tunnels, low-level regulated and unregulated outlets, a gated spillway at the HAD, and the Toshka flood release canal from Lake Nasr some 200 km upstream.

In determining the design flood discharge for the sluiceway at Naga Hammadi barrage two different approaches were considered. The first was related to the 'emergency release' from the HAD, and the second to the discharge from HAD based on routing of floods through the reservoir. The latter required an evaluation of extreme inflow floods to Lake Nasr coupled with a review of release strategies and operation of the various outflow structures mentioned above.

### 2.3.1 Flood Inflow Estimation to Lake Nasser

Flood inflow hydrographs to the HAD were derived for a range of probabilities of exceedance using statistical flood frequency analyses. Described in detail in Appendix J, they were based on the long-term inflow sequence (112 years) available from the Water Master Plan Study. Annual peak discharges and average inflows for a range of months were derived in order to determine both the design inflow peak and also 'shape' of the hydrograph for exceedance probabilities varying from 0.02 to 0.0001 (equivalent to recurrence intervals of 50 and 10,000 years respectively). The flood

estimates were subsequently modified to take into account existing levels of development upstream of the HAD, most notably the abstractions in the Sudan and losses in the Gebel Aulia.

A summary of estimated design flows for a range of probabilities of exceedance and duration are presented in Table 2.3.

Table 2.3: INFLOW FLOOD PEAKS AND FLOOD HYDROGRAPH VOLUMES TO LAKE NASSER FOR GIVEN RECURRENCE INTERVALS

| Recurrence Interval year | FloodPeak $m^3/s$ | Annual Flood Volume $10^9 m^3$ |
|---|---|---|
| 50 | 12,200 | 103.7 |
| 100 | 12,900 | 109.6 |
| 1,000 | 14,950 | 126.9 |
| 10,000 | 17,000 | 143.8 |

### 2.3.2 Assessment of the Function of Toshka Flood Release Canal

The combined discharge of the gated spillway and low-level outlets at HAD, which allow the turbines to be by-passed during emergency releases, would result in an outflow downstream to the River Nile considerably in excess of the `emergency discharge' of 7,000 $m^3/s$ defined by the MOPWWR. Due to the inherent risks associated with the occurrence of extreme floods, the Toshka flood release canal was constructed to discharge to the Toshka Depression west of Lake Nasser when water levels exceed a level of 178 m asl. This is equivalent to the crest level of the emergency spillway adjacent to the main embankment at HAD. A description of the Toshka flood release canal, its probable integrity under flood flow conditions based on a site visit, and a review of its discharge capacity are presented in Appendix I.

The Toshka flood release canal comprises an inlet structure located at Khor Toshka. The discharge passes through a canal which is terminated by a drop structure 20.4 km downstream. This comprises an end sill with a crest elevation at 176 m asl and a 25 m long low level apron at 172.5 m asl which terminates in baffles. The downstream canal discharges directly to the Toshka Depression. Its width varies between 350 m and 750 m.

The canal had not operated prior to 1996. After a series of above average inflows through the 1990s, the reservoir level increased above 178 m asl in at the start of November, 1996 and the canal has flowed continuously to the present (February, 1997).

The design discharge capacity of the canal was reviewed in the Feasibility Study using hydraulic backwater calculations on the basis of the surveyed 'as-built' canal cross-sections and dimensions of the end sill ogee. These indicated that the end sill does not control the discharge capacity. Rather, it is determined by the uniform flow conditions in the 20.4 km long canal. On the basis of likely hydraulic characteristics of the canal, the discharge at a reservoir level of 183.0 m asl, corresponding to the maximum allowable flood level for the HAD, is estimated to be approximately 2,800 $m^3/s$. This is some 10% lower than the discharge on which the design was developed.

Based on a site inspection, it was concluded there would be considerable erosion in sections of the canal and downstream sill in the event of a high discharge but it is considered unlikely that the integrity of the facility would be impaired (see Appendix I).

### 2.3.3 Release of Flood Inflow from Lake Nasser

A number of outlets within the HAD can be utilised to discharge under periods of more extreme inflow. These include:

i. six power tunnels feeding
   - 12 turbines,
   - 12 gated (ie regulated) low-level by-pass tunnels, another 12 unregulated low-level outlets which become operational after removing plugs,

ii. gated overflow spillway, and

iii. Toshka flood release canal.

The relatively long period over which the increase in flood discharge to Lake Nasser can be monitored, coupled with the capability to control the rate of discharge through the power tunnels and gated spillway, provide significant capability to regulate emergency releases from HAD.

To assess the magnitude of flood discharges from Lake Nasser a reservoir flood routing model was applied to determine the variation in reservoir levels and outflows under alternative release scenarios. Two cases were considered. The first was associated with routing of the design inflow flood to the HAD with a probability of exceedance of 0.0001 (10,000 year flood). The magnitude of the design flood during construction was also considered. A detailed description is given in Appendix J. The second related to the need to draw down the reservoir to an operating level of 150 m asl in case of an emergency.

*Routing of Design Inflow Floods*

Described in Appendix J, the magnitude of the release to the River Nile can be controlled under most conditions through operation of the various outlet facilities at the HAD, the main constraint being that the maximum reservoir level must not exceed 183 m asl. A series of flood routing simulations were therefore undertaken based on the estimated inflow hydrographs of various probabilities of exceedance as defined in Table 2.2. The reservoir was assumed to be at full supply level (178 m asl) at the commencement of the simulations. Alternative simulations were undertaken with discharges to the River Nile ranging from 5,000 to 7,000 $m^3/s$ (through operation of the outlet facilities). The peak reservoir levels and discharge through Toshka flood canal were noted for each case. Based on a compromise between the acceptable duration of very high releases downstream and the requirement to ensure the reservoir did not exceed 183 m asl, a flood release to the River Nile of 5,700 $m^3/s$ commencing in late August and continuing until the end of November was adopted. The associated maximum reservoir level was 182.96 m asl. The maximum discharge in Toshka flood canal was estimated to be approximately 2,800 $m^3/s$.

This flood release, which is based on a low probability inflow to Lake Nasser with a recurrence interval of 10,000 years and also implicitly considers the operation of HAD, is considered appropriate for the design of structures downstream. During this inflow flood, the discharge downstream would be maintained at a maximum of 5,700 $m^3/s$ for a period of 3.5 months and the reservoir would not exceed the maximum operating level of 183 m asl.

Similar analyses for inflow floods to the HAD of higher probability were also undertaken to define the magnitude of the diversion flood. Based on the 100 year inflow, a release of the order of 2,900 $m^3/s$ maintained throughout the year would result in a maximum reservoir level of around 181.7 m asl and also enable the reservoir level to be lowered to 175 m asl (additional criteria required by the MOPWWR) by the beginning of August the following year. On this basis, it is recommended that the diversion canal for construction of the New Naga Hammadi Barrage be designed to discharge 2,900 $m^3/s$.

### 2.3.4 Emergency Releases from HAD

In the case of emergency releases, simulations were undertaken for a range of target outflow discharges from 7,000 to 11,000 $m^3/s$. The analyses considered likely operating strategies of the HAD Authority including a maximum operating level at 1st August of 175 m asl. The inflow hydrographs were based on dry, average, and wet years. In addition, the commencement of releases was varied to assess the impact of delays in the implementation of the release decision, alternatively assuming 1st August, 1st September, and 1st October.

The flood routing studies based on a wet year inflow flood and the commencement of releases on 1st August indicated the required periods to lower the reservoir level to 150 m asl were around 8.6 and 6.6 months assuming emergency releases of 9,000 and 7,000 $m^3/s$ respectively. The latter is equivalent to the prescribed 'emergency release' for the Nile River. At a lower discharge of 5,000 $m^3/s$, the reservoir could not be drawn down prior to the onset of the high flows in the following year. Durations required under average and dry inflow years are marginally less.

If releases were to be initiated when existing reservoir levels were at a lower level, comparable to the average observed at the HAD immediately prior to the period of high inflow (July to September), the durations to draw down the reservoir are much lower. For example, under an average inflow, the drawdown would be completed in 1.3, 3.6, and 5.9 months for emergency releases of 9,000, 7,000, and 5,000 $m^3/s$.

The emergency release from HAD is defined in the reservoir operating manual as the release necessary to draw down the storage to the minimum operating level as rapidly as possible. It is the existing criteria for the design of hydraulic structures on the River Nile. As indicated above, when drawdown is initiated during the inflow flood season, the minimum operating level is reached after a period of some 6 months for the emergency release of 7,000 $m^3/s$. If initiated immediately prior to the flood inflow, the required duration reduces considerably to around 3 months.

## 2.4 HYDRAULIC ASSESSMENT OF RIVER REACHES UP- AND DOWNSTREAM OF NHB

Hydraulic investigations of the river reaches upstream and downstream of the NHB were undertaken to assess the impact of the New Barrage. These studies were carried out for the Consultant under separate subcontract by the Nile Research Institute (NRI) during the Conceptual Study (NHBDC, 1993) and partially by the Consultant.

### 2.4.1 Water Surface Profiles from NHB to Esna

Calculation of water surface profiles along a 177 km reach upstream of Naga Hammadi Barrage to Esna was required in order to assess the extent and magnitude of the backwater influence resulting from the present NHB headpond level, which varies seasonally from 65.10 m asl during the low flow to 65.40 m asl during the high flow, and the future headpond level of 65.90 m asl. The HEC-2 water surface profile model was applied for this purpose.

The calibration of the model was performed on the basis of water level measurements made at the eleven water level recording stations in the reach between Esna and Naga Hammadi for a range of ten-day mean discharges recorded downstream of Esna Barrage. These were 648 $m^3/s$, 1,736 $m^3/s$, and 2,629 $m^3/s$, representative of the minimum, mean, and maximum discharges over the ten years from 1983 to 1992. Although these vary from the discharges now observed (compare with Table 2.1) the calibration remains valid and has no impact on later applications of the model using a range of alternative discharges.

The cross-sectional data were based on the hydrographic surveys described in Section 2.1. The discharges were held constant over the full reach length, an assumption also adopted for subsequent surface profile calculations under the alternative design discharges and existing and proposed headpond levels. This is considered acceptable as the majority of abstractions from this reach occur immediately upstream of Naga Hammadi Barrage within the section where river water levels are comparable to the headpond.

For each discharge, Manning's 'n' values were varied to minimise the differences between simulated water surface profiles and those based on observed water levels along the total reach. The average roughness coefficient was 0.022 (standard deviation of 0.004) ranging from 0.0185 to 0.034. On this basis appropriate sets of 'n' were defined for three discharges ranges, namely up to 900 $m^3/s$, 900 to 2,200 $m^3/s$, and above 2,200 $m^3/s$. Verification of the model was performed for alternative periods in which the flows were comparable to those of the calibration period and for which water levels were also available.

Estimated water surface profiles for seasonal discharges (as defined in Table 2.2) and associated headpond levels at the existing Barrage (65.4 m asl in summer and 65.1 m asl in winter) were then derived for the upstream reach based on the design discharges presented in Table 2.2.

In evaluating the water surface profiles associated with the New Barrage, the abstractions immediately upstream of the existing Barrage must, however, be considered over the intervening length between the structures. Recent records indicate these are of the order of 200 $m^3/s$ during summer and 140 $m^3/s$ in winter. (Figure 2-1) Water surface profiles were then derived for the New Barrage with a headpond level of 65.9 m asl. The discharges assumed for the river upstream of EL Derb some 16 km upstream of the existing Barrage were the same as for the existing Barrage (this being the upstream limit of the major offtakes near Naga Hammadi). Downstream to the New Barrage they were reduced according to the irrigation abstractions. A summary of the predicted differences in river levels upstream of the existing Barrage are presented in Table 2.4. The calculated backwater profiles for the various discharges and headpond level of 65.9 m asl are presented in Figure 2-3.

Table 2.4: <u>BACKWATER LEVEL DIFFERENTIALS FOR HEADPOND LEVEL 65.90 m asl AT THE NEW BARRAGE</u>

| Location | River Kilometre from Aswan | Predicted Water Level Increases (m) Compared with Headpond Level 65.1 (1) | | Predicted Water Level Increases (m) Compared with Headpond Level 65.4 (2) | |
|---|---|---|---|---|---|
| | | River Discharge $m^3/s$ | | River Discharge $m^3/s$ | |
| | | 1,340 | 1,600 | 2,370 | 2,500 |
| Luxor | 223.80 | 0.00 | 0.00 | 0.01 | 0.01 |
| Shanhoria | 245.93 | 0.01 | 0.01 | 0.03 | 0.03 |
| El Balas Drain Outlet | 270.00 | 0.02 | 0.03 | 0.06 | 0.05 |
| Qena | 287.99 | 0.13 | 0.14 | 0.14 | 0.13 |
| El Samata | 295.12 | 0.21 | 0.23 | 0.18 | 0.17 |
| Dandara Irrigation Pumpstation | 298.50 | 0.24 | 0.24 | 0.20 | 0.19 |
| El Marashda Irrigation Pumpstation | 310.0 | 0.39 | 0.38 | 0.29 | 0.28 |
| Deshna | 317.75 | 0.48 | 0.46 | 0.35 | 0.34 |
| Hammad Outlet Drain | 331.00 | 0.59 | 0.57 | 0.43 | 0.42 |
| Derb Irrigation Pumpstation | 343.50 | 0.66 | 0.64 | 0.46 | 0.45 |
| Naga Hammadi Town | 346.55 | 0.68 | 0.66 | 0.47 | 0.46 |
| El Rawy Drainage Pumpstation | 349.00 | 0.72 | 0.70 | 0.48 | 0.47 |
| Existing Naga Hammadi Barrage | 359.50 | 0.82 | 0.82 | 0.54 | 0.54 |

Note: (1) Discharge characterising the low flow season

(2) Discharge characterising the high flow season

### 2.4.2 Water Surface Profiles from Assiut to NHB

The river channel downstream of the NHB has been subject to both local scour in the immediate vicinity of the Barrage and degradation/aggradation throughout the reach to Assiut since commissioning of the HAD. In deriving the tailwater rating curve at the New Barrage, it was essential that this be taken into account. As a result, the HEC-2 model was applied to derive a series of water surface profiles, and hence water depths immediately downstream of the barrage, for the full range of discharges to be considered in the engineering design. This included flows up the emergency discharge of 7,000 $m^3/s$.

Calibration of the model followed a similar approach to that adopted for the upstream reach. Firstly, concurrent water level measurements for a range of ten-day mean discharges of 744 $m^3/s$, 1610 $m^3/s$, and 2345 $m^3/s$ were obtained for the ten available recording stations between Assiut and Naga Hammadi Barrages. In addition, cross-sectional river channel profiles derived from the hydrographic survey information outlined in Section 2.1 were utilised. The Manning's n values were then varied to obtain the most accurate prediction of the observed water surface profiles. The average roughness coefficient was 0.021 with a low standard deviation of 0.004, similar to the river reach upstream of NHB. A recalibration using the program option to vary 'n' with discharge or water level was applied and sets of 'n' values were established for three ranges of discharge, namely up to 900 $m^3/s$, 900 to 2,200 $m^3/s$, and in excess of 2,200 $m^3/s$.

The simulations were applied for discharges from 350 $m^3/s$ to 7,000 $m^3/s$ and an equation defining the stage-discharge relationship downstream of the New Barrage, the tailwater rating curve, was derived as follows:

$$WL = 0.1166 * Q^{0.5132} + 55.50$$

or

$$Q = 65.8555 * (WL - 55.50)^{1.94856}$$

where

WL = tailwater level 200 m downstream of NHB, m asl
Q = discharge 200 m downstream of the axis of the New Barrage, $m^3/s$.

This curve is shown on Figure 2-4.

### 2.4.3 River Degradation Downstream of NHB

Tailwater levels will be directly affected by any future degradation in the reach downstream of the New Barrage, and particularly in its immediate vicinity. Detailed monitoring indicates river water levels have fallen significantly over the full range of discharges since implementation of the HAD. Hydrographic surveys along the River Nile have also indicated a change in bed levels, although results presented by the MOPWWR (1991) suggest the majority of this reduction occurred immediately after its construction.

At the Naga Hammadi Barrage, the drop in water levels in the first 5 years after commissioning of the HAD was some 0.4 to 0.5m, but the rate then reduced significantly and water levels dropped only a further 0.9m in the intervening years 1991. More recently, over the last 8 to 10 years, there has been little or no reduction in water levels over the full range of observed discharges. Nonetheless, prediction of future conditions downstream of the NHB, particularly any further degradation which might occur during the economic life of the project, was considered necessary. Such changes are of consequence for navigation, turbine setting height, structural stability of the barrage components, and energy generation. For this purpose, the HEC-6 one-dimensional mathematical model was applied (NRI, 1993) to analyse degradation and sediment deposition in the river reach between Assuit to Naga Hammadi Barrages. This model simulates the interaction between

water-sediment mixture, sediment material forming the stream's boundary, and the hydraulics of steady-state river flow.

Application of the model involved calibration using water surface profiles similar to the approach described for the HEC-2 simulations. In addition, parameters defining sediment properties and transport functions were also required. In previous modelling of the Nile River, Yang's stream power function has been found to most accurately simulate sediment transport and was therefore adopted for these analyses. Calibration of the model was based on an 11 year period of flows from 1982 to 1992 for which hydrographic sections were available from the bathymetric field surveys (see Section 2.1). The comparison of the observed and simulated longitudinal river bed profiles at the end of the simulation period (1992) showed good agreement.

Simulations to assess the long-term impacts of sediment transport within this reach and its effect on water levels immediately downstream of the New Barrage were then undertaken. Based on a 50-year period and adopting a 3-month time step, the long-term changes in river bed profile downstream of NHB to Assiut Barrage was calculated. For the purpose of the simulation, flow conditions observed through 1992 were applied. The results, like those of other similar degradation studies of the River Nile (MOPWWR, 1992), showed degradation for a limited distance downstream of the existing Barrage followed by minimal aggradation. The latter was only of the order of 0.2m and commenced 120 km downstream of the NHB.

Finally, utilising the long-term bed profile and discharges of 810 $m^3/s$, 1,600 $m^3/s$, and 2,500 $m^3/s$, estimates of the corresponding long-term water levels immediately downstream of the New Barrage were derived. The simulated drop in the water level is estimated to be some 0.7 to 0.8 m although this is possibly over-estimated. The estimated long-term tailwater rating curve based on these analyses is:

$$WL = 0.0879 * Q^{0.5463} + 54.90$$

The variables are defined above. This relationship was applied to determine the turbine setting height and for river navigation in the design of the project.

### 2.4.4 Determination of Management Lines

The MOPWWR is responsible for the control of all development and activities related to the river Nile within Egypt. In the late 1940s the Ministry established the concept of 'training' lines to define a right-or-way defined by the boundaries of the rive surface for a peak discharge of 11,000 $m^3/s$ over which it had jurisdiction for planning approval and overall responsibility. Following construction of the HAD, the flow regime and relevant design discharges altered significantly and the need to review these training lines became apparent. Rather than simply prepare new lines it was proposed by the RNDP (MOPWWR, 1990) that the concept of management lines be adopted.

Management lines comprise two components, namely *river channel lines* which correspond to the conveyance section required to discharge the peak irrigation release from the HAD as stipu ed by the MOPWWR (3,000 $m^3/s$), and *terrace lines* which convey the peak emergency release f. .n the HAD (7,000 $m^3/s$). The latter corresponds approximately to the limits of the old Nile flood plain and therefore will vary from these only where morphological changes such as erosion force their deviation in a landward direction. In effect, the definition of management lines creates three classes of land:

(i) areas outside the boundary of the terrace lines which are normally not within the control of the MOPWWR,

(ii) land between the river channel and terrace lines which could be released for low intensity use such as lowland agriculture, fish farming, temporary structures, and recreational activities,

(iii) land inside the river channel lines which should normally be held by the Government with its use only permitted after exhaustive study and approval by the MOPWWR.

In this Study, the establishment of the management lines (terrace and channel lines) for the upstream reach between Esna and Naga Hammadi Barrages was based on the water surface profiles related to discharges of 3,000 $m^3/s$ and 7,000 $m^3/s$ for headpond levels at the existing Barrage of 65.9 m asl and 67.5 m asl. (The studies were undertaken at an early stage during the Conceptual Phase before more detailed designs were available.) The results for the lower discharge are directly applicable to the New Barrage. The estimated water level upstream of the existing Barrage based on backwater analyses for the higher discharge and New Barrage was 67.33 m asl. This marginal difference with the adopted values of 67.5 m asl is negligible.

A complete set of the 1:10,000 scale maps on which the management lines are marked is presented in the associated report by the RNDP (1993). Based on these estimates, the area located between the terrace and river channel lines was determined. The results are presented in Table K5, Appendix K. The total area between the two lines is approximately 24.4 $km^2$ (5,800 feddans) for a headpond level of 67.5 m asl.

### 2.4.5 Possible Effect Of Upstream Abstractions By Toshka Pumping Station

A tender is currently underway for a new development being considered by the Egyptian Government which will involve pumping water directly from Lake Nasser just north of the Toshka emergency spillway. The proposed maximum discharge of the pumping station will be 300 $m^3/s$ but the actual discharge would vary seasonally in response to agricultural requirements. A preliminary assessment of the impact of this abstraction on future downstream releases from HAD was evaluated in order to quantify the affect on long-term energy generation at the New Naga Hammadi Barrage.

As it is likely that the 'New Valley' project will be expanded in line with agricultural, domestic and industrial development in the area, the maximum 300 $m^3/s$ discharge was assumed to occur only after 20 years. The peak discharge at implementation (assumed to coincide with the New Barrage at Naga Hammadi) was adopted as 90 $m^3/s$. This was assumed to increase to 160 $m^3/s$ and 230 $m^3/s$ after 5 and 10 years respectively in line with possible rates of development. The seasonal distribution of abstraction by the Toshka pumping station was then based directly on the observed releases into Western Naga Hammadi canal (immediately upstream of the existing Barage from which annual series of 10-day average flows were derived for each maximum discharge).

The annual energy generation for the New Barrage at Naga Hammadi for the adopted 4 unit design assuming development of the New Valley project is estimated to reduce by 0.5% at the start of operation of the pumping station, this reduction increasing to 3.3% after 20 years. This reflects the change in flows in the River Nile which, in comparison to those now observed at Naga Hammadi, would reduce considerably by 4.5% and 15% for these same periods.

The increased pumping during summer will actually result in higher power and energy generation at Naga Hammadi during this season than would occur with no development in the 'New Valley'. This energy gain results from the flows remaining higher than the design discharge of the powerplant, but due to lower tailwater levels the net head increases. However, this energy gain is more than offset by the overall reduction in flows and energy generation during the remainder of the year.

The present values of energy were also computed for both the with and without 'New Valley' scenarios based on a 50 year period commencing in the year 2006, which corresponds to the year of

implementation of the New Barrage. For a discount rate of 6%, the ratio of the respective present values of energy was 0.984. Overall, the proposed development is therefore likely to have only minimal impact on the energy generation for the New Barrage and negligible impact on the overall economics of the New Barrage hydropower development.

## 2.5 REGIONAL GEOLOGY

### 2.5.1 Morphology and Geological Setting

The valley of the River Nile in Upper Egypt is cut up to 500 m deep into Tertiary and Cr .aceous rock formations. The width of the valley varies locally, but generally broadens in the northward course of the river, being some 8 km to 12 km wide at Aswan and more than 35 km wide near Al Minya. In the project area, the average width is around 18 km with a narrow point of 13 km at the existing Naga Hammadi Barrage. Here, the river valley extends only to the south-east, while a steep limestone cliff rises up a few hundred meters directly beside the river bank.

The rock forming the valley boundaries consists of an Eocene limestone, the Thebes formation, with an exposed thickness exceeding 300 m. It consists of hard and bedded limestone and chalk (Lower Eocene) which is fissured and shows a certain degree of paleokarstification. A sequence of Paleocene and Cretaceous shale, marl and limestone with a thickness of some 400 m follows, with phosphates and oyster limestones at the base. The underlying Nubian Sandstone and basement rocks (Granite) are not exposed within the larger project area.

In the river valley, quaternary sediments with a thickness exceeding 100 m have been deposited by the river. At their surface, seasonal floods have created a fertile cover consisting of Holocene sandy and silty clay which forms the agricultural land. Its thickness varies up to 20 m near the riverbanks and runs out at the valley boundaries.

Below this top soil cover, Pleistocene and Pliocene sands and gravels form the main aquifer in the Nile valley. This formation is intensively layered, with a varying composition of sands, gravels or silty sands with lenses or layers of silt and clay. Since such river deposits were sedimented during a continuous change of the river morphology in history, the vertical sequence and horizontal extension of the stratification is often irregular.

### 2.5.2 Seismicity

According to the "Earthquake Hazard Atlas", Egypt is considered to be a country of low to moderate seismic hazard. Situated along the NE margin of the African plate, as shown on Figure 2-5, earthquakes result mainly from the tectonic structures along this plate margin.

For the purpose of earthquake hazard assessment, Egypt can be subdivided into areas characterized by the rate of seismic activity. As a broad generalization, the northern part is tectonically unstable, whilst the southern part is classified as stable. Furthermore, the area east of the Nile River is relatively unstable and marks the transition to the Red Sea Rift, whereas the westerly sect ns are stable and form part of the African massif. Hence, the north-east part of the country is potent .ly the most hazardous, and the south-west part the least hazardous.

This seismic risk assessment is in line with the record of 257 events at various distances from the Naga Hammadi Barrage (Figure E-7, Appendix E) in which more than 95% are located in the northeast.

According to the Munich Insurance Company (Germany), the Naga Hammadi Barrage is located in a zone where an earthquake of intensity VI (MM) has a corresponding return period of 270 years. The associated acceleration at the epicenter is 0.035g to 0.07g, supporting the more regional seismic risk

assessment outlined above. Furthermore, a number of conservative evaluations based on the available seismological data confirmed rather low design accelerations of less than 0.1g.

In summary the design earthquake acceleration at the New Barrage will not exceed some 0.07g. Thus, the seismic environment will not significantly affect the structural design. According to engineering practice, for accelerations of that order of magnitude sufficient seismic stability is already directly considered when determining the structural stability with the appropriate factors of safety.

## 2.6 GROUNDWATER INVESTIGATIONS AND MODELLING

The New Barrage will be constructed with a design headpond level of 65.9 m asl, equivalent to the design headpond level of the existing barrage. The recent operation of the Barrage has seen the headpond level vary seasonally with a summer level of 65.4 m asl (June to August) and winter level of 65.1 m asl (apart from a brief period during winter closure). The headpond level of the New Barrage will therefore be increased by 0.5m and 0.8m above respective seasonal levels. This increase is likely to result in changes in the groundwater levels in the underlying aquifer system both upstream and downstream of the barrage with a consequent impact on the local environment. In order to quantify the magnitude of these impacts, groundwater investigations were undertaken. These comprised both the initiation of a groundwater monitoring programme incorporating an extensive network of new piezometers drilled during the Feasibility Study and the application of a mathematical groundwater model to simulate the groundwater system under existing and higher headpond conditions. Details are presented in Appendix M.

### 2.6.1 Available Data and Description of Aquifer System

The local stratigraphical succession in the Project Area can be characterised as comprising (from ground surface down):

- The **Holocene** silty clay, intercalated with gravel and sand varies from zero to some 20m in thickness. The fertile agricultural land is formed by this unit.

- The sands and gravel of the **Pleistocene** form the main aquifer in the Naga Hammadi area. The thickness, mainly elaborated from geoelectrical surveys, varies from 40m to 230m.

- The main aquifer overlies several geological units which are interpreted as aquicludes for overlying aquifers or secondary aquifer systems.

On the basis of geotechnical analysis of samples from boreholes drilled in the near vicinity of the barrage and piezometers installed more widely throughout the project impact area (see Appendix M) the average permeabilities of the aquifer system over the entire Project Area can be summarised as follows:

- In the upper **Holocene** layer, horizontal permeabilities ($k_h$) are likely to be within the range of $3x10^{-4}$ to $9x10^{-4}$ m/s. The vertical permeability ($k_v$) is estimated to be within the range of $4x10^{-7}$ to $2x10^{-8}$ m/s.
- In the lower **Pleistocene** layer (main aquifer), horizontal and vertical permeabilities are estimated to vary within a range of $1.4 \; x10^{-3}$ to $9x10^{-4}$ m/s.

Additional data required for the analyses, details of which are presented in Appendix M, included:

- contour information defining ground surface levels within the study region,
- details on local infrastructure,
- hydrogeology to define model parameters for groundwater flow,

- water levels of the River Nile and irrigation and drainage canals and discharges for the latter,
- hydrometeorology including local rainfall and evapotranspiration, and
- data on locations and rates of groundwater abstraction.

Much of this information, including irrigation and drainage systems, local infrastructure, borehole locations, and points of groundwater abstraction, is shown schematically on Album Nos. 87 and 88. In addition, information from the previous groundwater modelling studies undertaken during the Conceptual Phase was also considered.

For the purpose of measuring the immediate response of the aquifer system to irrigation application and seasonal changes in the water levels of the River Nile, major emphasis was given to the establishment of an extensive groundwater monitoring programme during the Feasibility Study. This extended over the wider area which could be affected as a result of increased groundwater levels resulting from a higher headpond. The programme comprised the installation of a total of 53 piezometers. The majority extend on average 5 to 10 m, into the upper **Holocene** aquifer. A smaller number extend 15 to 20 m, into the lower **Pleistocene** aquifer, to monitor the impacts of deeper infiltration to the underlying aquifer which is also utilised for local groundwater abstraction. The boreholes extended from locations 65 km upstream to 20 km downstream of the New Barrage and were distributed across the valley to the extremities of the existing irrigation area.

The new observation programme commenced in April, 1995 and is now maintained on a continuous basis in accordance with an established program. This entails weekly observations of all boreholes to the north of Naga Hammadi town and two weekly observations of those to the south. From June to July, 1995, the boreholes were observed more frequently to note any variation in response to the period of higher irrigation application. The available data now encompass more than one hydrological year. The importance of this programme cannot be understated and it is strongly recommended that it be continued on an ongoing basis both up to and beyond construction of the proposed New Barrage. Based on the available observations the conclusions are:

- Groundwater movement in a zone approximately 1 km either side of the River is mainly influenced by the water level of the Nile River, with higher groundwater levels generally coinciding with the period of high summer flow. Significant drops of some 2m occur during winter closure.

- Areas located in a distance of more than 5 km from the river are mainly influenced by the amount of irrigation water. In these areas, the groundwater level maintain at relatively constant level despite the fluctuations of the river water level. Only during winter closure groundwater levels drop by some 1m to 2m.

- The majority of the project area is influenced by both the Nile River water level and the volume of irrigation water. Maximum groundwater levels usually occur during the period of high discharges coupled with maximum irrigation. The winter closure leads to a significant reduction in the groundwater levels of about 1m to 2m.

### 2.6.2 Groundwater Modelling

The study region is characterised by a complex network of irrigation and drainage canals (see Album No. 87 and 88), and hence a relatively complex surface water-groundwater system. Prediction of potential environmental impacts for such systems on the basis of estimated changes in groundwater levels can therefore most accurately be made on the basis of results from mathematical groundwater models of a suitably high technical standard. These results could then be used to assess the environmental impacts and develop appropriate methods to mitigate these affects. The associated costs would then be included in the financial costs for project evaluation.

*Description of the Model*

The model applied for the Feasibility Study was a quasi three-dimensional finite element approach based on a steady-state representation of conditions in the project area. A detailed description of the model, the hydrogeological and surface water parameters used for its calibration and verification, and results of the simulations based on the proposed development of the New Barrage are described in Appendix M.

The principal model parameters defined the hydrogeological characteristics of the upper and lower aquifer systems and surface water and other infrastructure components in the area with a direct hydraulic connection to the underlying aquifers. The major components of the latter group include the River Nile and irrigation and drainage canals with their constant heads and capacity to act as either sources or sinks in the system. Other factors which were also considered were the irrigation application rates, losses from the soil due to evapotranspiration, and groundwater abstractions (primarily from the lower **Pleistocene** layer) and the areal extent of subsurface tile drainage networks in the area.

Estimates of water levels in the river and irrigation and drainage canals used as model input in the calibration and verification (as described below) were based directly on historically recorded data. Predictions of groundwater flow and associated groundwater levels were validated based on observed groundwater levels of the recently established borehole monitoring network.

The adopted model boundaries (shown in Album Nos. 87 and 88) were initially established on the basis of the groundwater simulations undertaken during the Conceptual Phase and considering the results of the backwater calculations upstream of the New Barrage. The latter provided an indication of the upstream extent of significant changes in river levels as a result of the design headpond level for the New Barrage (see Table 2.4). The boundaries parallel to the river were determined largely by the existing structural plateaux and extent of the irrigated areas. The up- and downstream boundaries were located respectively some 65 km upstream and 20 km downstream of the New Barrage.

The validity of the upper limit was further assessed by comparing observed groundwater levels during the summer and winter seasons for the two most upstream boreholes, P1 and P2, located at the upstream boundary and 2 to 3 km from the river channel. The observed differences in groundwater levels for these boreholes from summer to winter were 0.21 and 0.28m respectively. The difference in water levels at the closest point of the river from both is considerably larger at 0.96m. This suggests relatively large differences in river levels in this area do not influence areas apart from immediately adjacent to the river. (This was not unexpected as the river generally acts as a sink apart from near the headpond area of the barrage.) As the differences in river levels between the existing and future condition are only around 0.2 m (and reduce quickly to less than 0.1 m a further 10 km upstream) this boundary is certainly adequate.

Downstream, the river levels are not influenced at all by the higher design headpond level of 65.9 m asl and the 20 km distance adopted is certainly adequate to ensure no groundwater related impacts resulting from the headpond itself.

*Calibration and Verification of Model*

Prior to applying the groundwater model for the prediction of groundwater levels under alternative headponds, a calibration and verification of the model parameters was undertaken. This involved simulation of the groundwater system and comparison of the observed and estimated groundwater levels during periods when all required data were available and the system was in a steady state in terms of variation in river discharges and barrage headpond level. The calibration, based on the period from 3$^{rd}$ to 11$^{th}$ May, 1995, included a progressive modification of the relevant model parameters to most accurately simulate both the observed piezometric levels and also the overall water balance. Permeabilities of both aquifers, vertical leakage, and leakage of canals and drains were varied within realistic limits.

The average difference between the estimated groundwater levels and those measured at the boreholes was only 0.04 m. The maximum variation was 0.35 m but only at three boreholes did the difference exceed 0.10 m. The results of the calibration run showing piezometric contours of the **Holocene** aquifer is shown on Album nos. 89 and 90.

In accordance with the Terms of Reference for the Feasibility Study, simulations were carried out for two representative seasons (summer and winter respectively). The verification periods were selected to ensure there was no transient behaviour of boundary conditions and the River Nile discharge and irrigation demand was in phase. This resulted in verification periods being defined for the summer season from **June** to **August** and winter from **February** to **April** and **November**. A summary of the boundary conditions is presented in Table 2.5.

Table 2.5: REPRESENTATIVE SEASONAL BOUNDARY CONDITIONS

| | | Summer | Winter |
|---|---|---|---|
| Water level River Nile u/s Barrage | m asl | 65.40 | 65.10 |
| Water level River Nile d/s Barrage | m asl | 61.63 | 59.97 |
| Discharge upstream | $m^3/s$ | 2,370 | 1,340 |
| Discharge downstream | $m^3/s$ | 2,170 | 1,200 |
| Total Irrigation Amount | $m^3/s$ | 62.20 | 38.83 |
| Average Drainage | mm/month | 50 | 33 |
| Evapotranspiration | mm/month | 213 | 130 |
| Groundwater Abstraction | $10^3\ m^3/d$ | 201 | 201 |

A comparison of the observed and estimated groundwater levels for the piezometers in the upper aquifer showed an average difference of 0.01 m for both seasons. The maximum difference was 0.24 m in summer and 0.22 m in winter. The groundwater levels for the summer verification period are presented on Album nos. 94 and 95, while for the winter season they are shown on Album nos. 96 and 97. The water balance of the overall system was assessed by sub-dividing the model area into five sub-areas generally on the basis of the existing irrigation areas as shown in Figure 2-6. Overall, the water balance for each sub-area resulted in realistic estimates for the principal model variables, particularly the drainage by main drains for both summer and winter season (see Appendix M, section 5.2).

*Simulations*

Following calibration and verification of the groundwater model, simulations of the system for the New Barrage with an increased headpond level to 65.9 m asl were undertaken for the same summer and winter seasons. The boundary conditions remained unchanged from those adopted during the verification runs. The simulations indicated that under present headpond conditions, in summer and winter the areas in which groundwater would be within 1.0m below the groundsurface are 51,676 and 38,898 feddan respectively.

The results as well as field inspections and inventories of the local irrigation and drainage system highlighted an area of major concern for the drainage capacities. Overall, the drainage system is poorly maintained at present with excessive growth of aquatic weeds resulting in far higher water levels in the drains than under normal operating conditions. In addition, under existing headpond conditions the drainage station at Hammad remains submerged for the majority of the year resulting in a backwater effect which extends a significant distance upstream in the main drain. In turn this reduces the drainage efficiency of this system.

The MOPWWR recognises the potentially negative impacts this is having on the system. Whilst it is preferable that the complete open drainage network within the project impact area be properly

maintained on a regular basis, the current poor condition of the network indicates such a programme has not been sustainable over recent years. As a result, it was agreed in conjunction with the MOPWWR that in assessing the impacts of the project on groundwater levels the following would be assumed:

i. There will definitely be an improvement in the maintenance of the open drain network at least to the stage of properly maintaining the most important open and secondary drains of the system.

ii. The improved maintenance should be introduced into the groundwater modelling for all simulations, that is for both the present conditions and following implementation of the project.

The locations where a sustainable maintenance programme of the open drains (involving regular and effective clearing) would be most beneficial was then developed on the basis of the results of the verification runs. The extent of the programme, which would directly reduce water levels in the targeted drains by between 0.5 and 1 m, totals some 129 km and is shown on Figure 2-7. In addition, a drainage pumping station was assumed for Hammad Drain outlet. (Discussions were held by the Drainage Directorate in Qena regarding proposals for its construction.)

Simulation runs were again undertaken assuming the improved maintenance of the open drains for the existing headpond conditions and for the New Barrage at 65.9 m asl and for each season respectively. The results are presented in Table 2.6 and shown schematically on Album nos. 97 to 104.

In addition to the maintenance programme of open drains, the Drainage Directorate is also upgrading existing tile drainage systems and installing new systems in the project area. Considering the ongoing and committed programmes to be completed by 1998, the incremental areas which would be affected by the project in the longer term are also summarised in Table 2.6. These highlight the areas with depths to groundwater of less than 1.0 m, 0.75 m, and 0.60 m which respectively relate to critical values associated with the infrastructure and summer and winter crops potentially at risk in the project area as outlined below:

| | |
|---|---|
| summer crops | 0.75 m |
| winter crops | 0.60 m |
| garden fruits | 1.00 m |
| sugar cane | 0.40 m |
| damp proof of houses | 1.00 m |
| non-isolated septic tanks | 1.00 m |
| graveyards | 0.50 m |
| historic monuments | defined individually. |

As some 70% of the irrigated area is used for sugar cane production (with a critical depth of 0.4 m) the results in Table 2.6 indicate no impacts are expected either at present headpond conditions or following project implementation. The affected areas with depths to groundwater of less than 0.75 m and 0.60 m in summer and winter respectively are given for the existing and New Barrages on Figure 2-7, showing all incremental changes also in the areas with ongoing or planned installations by the Drainage Directorate.

Table 2.6: ESTIMATES OF AREAS OF PROJECT IMPACT BASED ON GROUNDWATER SIMULATIONS

| Period (Nile River Discharge) | Depth to Groundwater less than m | Area Without Project | Area with Project feddan | Area Affected by Project feddan |
|---|---|---|---|---|
| Winter ($1,340\ m^3/s$) | 1.00 | 18,900 | 22,080 | 3,180 |
| | 0.75 | 6,430 | 9,475 | 3,045 |
| | 0.60 | 1,865 | 2,840 | 975 |
| | 0.40 | 0 | 0 | 0 |
| Summer ($2,370\ m^3/s$) | 1.00 | 29,805 | 32,475 | 2,670 |
| | 0.75 | 11,820 | 15,445 | 3.625 |
| | 0.60 | 5,545 | 6,585 | 1,040 |
| | 0.40 | 0 | 0 | 0 |

Note : Headpond level 65.9 m asl, tile drainage network including ongoing and planned tile drainage programmes

## 2.7 GEOTECHNICAL CONDITIONS AT THE PROJECT SITE

The geotechnical investigations undertaken during the Feasibility Study included drilling of boreholes in both the Barrage and downstream at the site of the New Barrage. A network of piezometers was also installed to monitor piezometric heads in the area. Those boreholes at the Barrage were drilled to assess the condition of the existing structure and underlying foundations and also the geotechnical conditions of the area in which a hydropower plant would be constructed. Downstream at the site of the New Barrage, the drilling was undertaken to assess foundation conditions. The drilling also served to determine the existence of an underlying clay layer which would provide a sealing layer suitable for the construction pit.

### 2.7.1 Scope and Results of Geotechnical Investigations

*Scope of Investigations*
In the vicinity of the existing Barrage, the following geotechnical investigations were carried out:

- Eleven boreholes were drilled through the piers and the bottom slab into the foundation soil and were equipped with piezometers, in order to investigate the condition of the piers and the foundation and to monitor the piezometric head under the barrage.
- Geophysical logs in three of the boreholes and continuous water pressure tests in six of the boreholes drilled through the piers in order to investigate the structural condition of the barrage.
- Four boreholes were drilled through the base slab upstream of piers and were equipped with piezometers to monitor the piezometric head.
- Four boreholes were drilled some 250 m upstream of the piers to investigate the sedimentation in the headpond.
- Nine boreholes were drilled downstream of the existing Barrage to investigate the geotechnical conditions for a construction pit and the foundation for a powerplant at the existing Barrage.

In the vicinity of the New Barrage and on El Dom Island, the following geotechnical investigations were carried out:

- A grid of 20 boreholes was drilled offshore and along both riverbanks adjacent to the New Barrage to investigate the geotechnical conditions for the construction pit and foundation.

- Seven boreholes were drilled on El Dom Island and in the floodway to investigate the geotechnical conditions for the construction pit and the foundation for a New Barrage located on El Dom Island.
- 41 Dutch Cone Tests were carried out along the river banks between the existing Barrage and the New Barrage to investigate the foundation conditions of the protection dykes.
- Two pumping tests were undertaken comprising the construction of two pumping wells and the installation of 18 monitoring piezometers on El Dom Island near the right bank to assess in-situ permeability coefficients.

The locations and depths of the boreholes are shown on Album No. 59 and in Table E-2, Appendix E. In all boreholes, Standard Penetration Tests were performed at intervals of 1.5 m to 3 m to determine the density of the soil and to take samples from the split-barrel. Comprehensive laboratory tests on these samples were carried out as described in Appendix E.

*Drilling and Sampling*
On the basis of the drilling and sampling during the geotechnical programme, the stratification described below was observed. The corresponding geological sections are shown in Album Nos. 60 to 64 and Table E-3, Appendix E.

The top soil cover consists of a clayey, sandy silt, or silt and clay mixtures (Holocene) which was encountered in all onshore boreholes except those in the floodway. The thickness varies mostly between 6 and 10m. Below the top soil cover, an alternating sequence of poorly graded sands (Pleistocene) is the dominant strata. The sequence comprises coarse to medium sands, medium sands, or fine to medium sands locally containing some fine gravel or silt. Deeper than some 30 m asl, sands with some silt are again encountered. At varying depths, sand-gravel mixtures with a thickness between a few decimeters and 10 m are interspersed. As no continuity of these sand-gravel mixtures can be identified, they are assumed to be local lenses. The sand formation forms the main aquifer over the entire project area. The groundwater levels on the riverbanks, on the island and in the floodway correspond closely to the river water level.

The formation of sand is also interspersed by layers of gravel with "broken stone fragments", varying in thickness between a few decimeters and several meters. In-situ grain sizes up to cobbles and boulders are expected. Such layers were encountered in two prevailing zones as shown, for example, on Figure 2-7 for the sluiceway at the New Barrage, namely:

- Gravel 1 with an average thickness of 2 m between 44 m asl and 38 m asl at the New Barrage and upstream but not continuing to the existing Barrage.
- Gravel 2 with an average thickness of 1 m between 24 m asl and 26 m asl at the existing Barrage, and between 26 m asl and 30 m asl at the New Barrage and further downstream.

Lenses of gravel with broken stones exist locally in a large variety between 45 m asl and 50 m asl, with thicknesses not exceeding 2 m.

Artesian water with a head of more than 4 m above river level was encountered offshore directly downstream of the Barrage. At a distance of some 2.5 km downstream, an artesian head of 2.8 m above river level was encountered in boreholes on the El Dom Island and upstream of the New Barrage.

Under the sand formation, a consistent layer comprising silt and clay with some fine sand was encountered only at the New Barrage and further downstream at levels between 24 m asl and 10 m asl. At the Barrage, this layer was not encountered consistently, so that only lenses of clay are assumed to be present. Further upstream of the New Barrage and in the remaining area of the island and floodway, no silt and clay layer was found. The minimum thickness of the silt and clay layer in the area of the future construction pit of the New Barrage is 5.40 m.

Samples taken from the silt and clay layer were investigated and classified geotechnically. According to grain size and plasticity, the majority of samples were clays with high plasticity belonging to the group CH and a few were clays with low plasticity of the group CL. The content of fines ranged between 45% and 95%. The natural water contents were mostly near the plastic limit, which implies consistency indexes around 1.0 (stiff to hard clay). The organic matter content was found to be negligible. The permeability is estimated to be well below $10^{-7}$ m/s.

Direct shear tests on sand samples resulted in friction angles between 31° and 38° with a cohesion of zero. For dense sand an average friction angle of 35° can be assumed; for loose or medium dense material, 32° is appropriate.

*Permeabilities*

As no geotechnical devices exist to obtain reliable samples from coarse gravel and cobbles deep below the groundwater surface (gravel 1 and gravel 2 as described above), a pumping test was carried out to determine geotechnical properties, in particular the permeability, of such layers. In addition, the permeability of an intensively layered sequence such as the sand formation in the Project Area should be determined in situ. For this purpose, pumping tests in two separate wells with filter sections in the gravel 1 exclusively, and over a 20 m thick section of the sand formation, were carried out. The water levels at various depths and distances from the wells were monitored by piezometers.

The records of the groundwater levels at various depths showed that heads exceed the hydrostatic level below the gravel layer. This increase appears consistently from the gravel 1 to the underlying sand and averages 0.6 m from 40 m asl to 33 m asl.

The steady state and transient evaluation of the pumping tests with a 3-dimensional groundwater model resulted in average permeabilities at the right bank of the New Barrage as follows :

| | | |
|---|---|---|
| sand above 40 m asl: | lateral permeability : | $1 * 10^{-3}$ m/s |
| | vertical permeability : | $2 * 10^{-5}$ m/s |
| gravel 1 at 40 m asl: | permeability : | $2 * 10^{-3}$ m/s |
| sand between 40 m asl and 33 m asl: | lateral permeability (m/s) : | $1 * 10^{-4}$ m/s |
| | vertical permeability (m/s) : | $1 * 10^{-5}$ m/s |

The results from the pumping tests were confirmed by a comprehensive series of grain size analyses on samples from the sand formation. These enabled both a detailed geotechnical classification of the soil within the project site and also the derivation of horizontal permeabilities within various depths. The results, described in Appendix E, are summarized below:

| | | |
|---|---|---|
| sand above 30 m asl: | lateral permeability (m/s) : | $2 * 10^{-4}$ m/s - $8 * 10^{-4}$ m/s |
| sand below 30 m asl: | lateral permeability (m/s) : | $2 * 10^{-5}$ m/s - $4 * 10^{-4}$ m/s |

### 2.7.2 Interpretation and Conclusions for the Design

From the drilling and laboratory investigations, it is concluded that a silt and clay layer is continuous over the entire area of the construction pit for the New Barrage. This layer is suitable to serve as a bottom seal for the construction pit if it is penetrated by a surrounding diaphragm wall. At the existing Barrage and in the area of El Dom Island and the floodway, no such layer can be expected.

The Standard Penetration Tests under the foundation slab of the existing Barrage indicated that the compaction of the foundation soil during construction was at best only limited. The loose to medium density measured corresponds to the natural density occurring at that elevation.

The Standard Penetration Tests in the boreholes at the New Barrage indicate at least medium density (navigation lock) or dense material in the foundation area of the major components of the structure.

From the field observations during drilling and pumping tests, and from the permeability estimates based on grain size analyses and the pumping tests, the following conclusions are drawn:

- Downstream of the existing Barrage, a quite consistent layer some 40 m below the surface and described as gravel 2 (Section 2.7.1), is subject to artesian pressure. The pressure corresponds with the u/s water level at the existing Barrage and reduces further downstream due to considerable vertical groundwater flow through the overlying sand.

- The permeability of the sand formation is characterized by a significant difference between horizontal and vertical permeabilities.

- The permeability of the gravel 1 (Section 2.7.1) is around $2 * 10^{-3}$ m/s.

Based on assessments of the borelogs, Standard Penetration Tests, Dutch Cone tests, the pumping tests, laboratory analyses and experience with similar soils, the stratification was grouped and geotechnical design parameters allocated as outlined in Figure 2-8. For specific features, reference is made to Appendix E and the geological sections, Album Nos. 60 to 64.

## 2.8 CONSTRUCTION MATERIALS

### 2.8.1 Requirements

For the construction of the New Barrage and appurtenant embankments, a summary of the material requirements is presented in Table 2.7.

Table 2.7: VOLUME OF REQUIRED CONSTRUCTION MATERIAL

| | Type of Material $m^3$ | | | |
|---|---|---|---|---|
| | Sand and Gravel | Filter | Rockfill | Rip rap |
| Right Bank Closure Dyke | 176,700 | 9,500 | - | 11,800 |
| Right Bank Dyke along Island | 39,300 | 6,500 | - | 8,200 |
| Left bank protection dyke | 168,500 | 12,000 | - | 15,000 |
| Diversion canal | - | 115,000 | - | 96,000 |
| Cofferdam u/s | 120,800 | 19,000 | 74,600 | 8,100 |
| Cofferdam d/s | 127,900 | 16,000 | 74,800 | - |
| Head- and tailrace channel | - | 226,000 | - | 455,000 |
| Closure dam | 165,600 | 1,800 | 154,900 | 2,200 |
| Total volume required | 798,800 | 405,800 | 304,300 | 596,300 |

### 2.8.2 Potential Sources and Suitability

The required volumes of sand and gravel will be obtained from site excavations which will far exceed requirements.

For rock material, four potential quarries and borrow sites were assessed, Esawia, Tafr and Rageh quarries and rock outcrops along the Naga Hammadi-Sohag road. Their locations, together with the locations of sand and gravel pits are shown on Figure 2-9. For each site, assessments were made on

the rock quality in-situ, the possibilities of quarry extension and increase of production. The Tafr and Rageh quarries have very limited potential for expansion and were therefore not regarded as suitable. The various rock outcrops inspected all consist of weak chalky limestones with abundant chert. The possibility of opening a new quarry at these rock outcrops is not considered feasible.

At Esawia quarry the overall quality of the rock is not constant within the same bank and probably varies also with depth. However, the Esawia limestone is regarded as suitable for rip rap and rockfill due to the low loads both will be exposed to, particularly if the rock is quarried at selected locations. The material quantities available from the quarry significantly exceed project demands.

The main source for concrete aggregates in the Project Area is the Al-Bosa gravel pit some 20 km to the north-west of the project site. In addition, the Hiw sand-pit was considered for the supply of coarse sand if this material is insufficient from Al-Bosa or the site. If coarse filters cannot be processed on site from excavation material, they will be borrowed from the Al-Bosa gravel pit. Details on the site inspections and results of laboratory investigations are explained in Appendix G.

# 3. PLANNING CRITERIA FOR THE NEW BARRAGE

## 3.1 GENERAL

The development of planning criteria for the Feasibility Study has been an evolving process throughout the Project.

Initial criteria were developed during the Conceptual Study which were commonly applied to the design of all layouts and their components, with the aim of ensuring comparison of designs and cost on an equivalent basis. The overall design of the project components was adapted from the corresponding components of the New Esna Barrage.

During the Interim Phase, and in line with recommendations by the POE, the sluiceway was redesigned with larger gates. Based on additional field investigations and studies, planning criteria were also augmented and more specifically developed to the design of the New Barrage.

As can be noted from the general layout shown in Album No. 7 and the geotechnical sections shown in Album Nos. 12 and 60 to 64 for the New Barrage, the site conditions differ substantially from those at Esna, thereby requiring a different approach for construction. In addition, a decision was taken at the Feasibility Stage that the powerhouse and navigation lock would be designed independently and not be based simply on a modification to the design from the New Esna Barrage.

The concept of safe evacuation of the emergency release from HAD was adopted during the feasibility study to the requirements of the MOPWWR, by which the navigation lock`s discharge capability is seen as an additional degree of safety only.

On request of the MOPWWR, the results of the optimization of the headpond level undertaken in the Interim Study were disregarded in the design headpond level for the New Barrage was set to 65.9 m asl. With the aim of avoiding any impact on the control of the irrigation system, the MOPWWR and the MEE agreed to discard the option of peaking operation for the powerplant and consider only run-of-river operation.

The re-distribution of the releases from the HAD over the year as already observed during the years 1994, 1995 and 1996 was regarded to be indicative of the future release pattern from the HAD. The minimum release during the winter closure period was decreased from 710 to 350 $m^3/s$. This required a lower setting of the bottom of the navigation lock and the approaches, and a lower setting of the generating units.

The decision as to whether to use the New Barrage for public road crossing of the River Nile in addition to the road on the existing Barrage will depend a decision by the MOPWWR. With the aim of providing the database for this decision-making, an assessment of cross river traffic is given in Appendix V.

Overall, the planning criteria applied for the New Barrage are therefore an amalgam of results from the engineering studies and field investigations during both the Interim and Feasibility Phases, technical recommendations by the POE, and the operational requirements finally introduced by the MOPWWR.

## 3.2 PLANNING CRITERIA

The criteria adopted for the design of the New Barrage are outlined below.

1. <u>Headpond Level</u>

Subsequent to the optimizations made in the Interim Study and re-evaluations during the Feasibility Study including the assessment of environmental impacts due to an increased headpond level and its

effect on groundwater levels, the MOPWWR decided that the headpond level of the New Barrage shall be at 65.9 m asl, which is equivalent to the design level of the existing Barrage.

The MOPWWR further stated that during operation the headpond level of the New Barrage will be maintained at 65.9 m asl for at least 90% of the time, but during some 30 days per year the headpond level may be lowered within a range of up to 0.5 m. This temporary, short-term reduction is proposed to meet short-term increases in the irrigation demand downstream of Naga Hammadi. The significant distance from the HAD to the irrigation offtakes serving the areas of increased demand, and hence time lag in water reaching these areas, precludes they being met quickly by an increased release from the HAD (see Appendix H).

2. Installed Capacity

In the Conceptual and Interim Phases, the installed capacity was assumed to correspond to a maximum turbine discharge of about 2,000 $m^3$/s. At feasibility study level, the installed capacity is determined by economic criteria, in which the incremental energy cost is limited by the cost of equivalent thermal generation. Details of the optimization of the installed capacity (or maximum turbine discharge) are contained in Appendix Q. The resulting maximum turbine discharge is 1,840 $m^3$/s, corresponding to a river discharge with 24.5% exceedance, and the powerhouse design discharge is 1,280 $m^3$/s. The optimization was based on a powerhouse with four generating units each comprising three blade bulb turbine units.

3. Type of Generating Units

Propeller and Straflo units, being both single-regulated turbines with fixed runner blades, are not a suitable solution for the Naga Hammadi project because of their hydraulic characteristics. The shape of the efficiency curves of these types of units are such that high efficiencies can be achieved only within a narrow range of flows and heads and the efficiency deteriorates rapidly outside that range. The wide range of operating flows and net heads at Naga Hammadi cannot be reasonably covered by 4 or 6 units with fixed blades.

Hence, double regulated bulb turbines with directly driven generators are the correct choice for the head and flow conditions in the River Nile at Naga Hammadi. Pit turbines with the generator driven through a speed increaser would have lower reliability and lower overall efficiency.

With the aim of minimising cost and maximizing energy generation, both three and four blade bulb turbines were under investigation. While the four blade units reach generally higher efficiencies over the entire range of operation, their discharge capacity above design discharge is significantly smaller than that of a three blade unit of the same diameter. With the same diameter and at lower cost, the three blade units can operate over a larger range of the discharges during the high flow period than would be possible with four blades. As the energy generation with the three blade units is higher than with four blade units of identical size, see Appendix O, the three blade bulb turbines were selected.

4. Number of Generating Units

For the range of powerhouse design discharge used in the optimization, namely 1,050 to 1,650 $m^3$/s, consideration was given only to an even number of units (either four or six) which facilitates the economic arrangement of the main transformers and electrical auxiliaries serving two generators.

The average generation performance (see Appendix O) shows a difference of only 2 GWh/year in favor of the four units when compared with six units of equal discharge capacity. The combined cost of the civil works and the mechanical and electrical works for the range of units considered indicates marginally higher costs for the six units.

Consideration of the incremental cost of generation (see Appendix Q) over the entire range of discharge capacities in the optimization (Appendix Q) revealed production cost increments for the six unit powerhouse being clearly above critical limits when compared with alternative thermal generation. Hence the four unit powerhouse with three blade units was selected.

5. Bulb Turbine Rating and Setting

The four turbines were optimized for the following operating conditions:

- Total powerhouse design discharge, 4 x 320 $m^3/s$: $Q_t$ = 1.280 $m^3/s$
- Corresponding net head over the sluiceway: $H_n$ = 5.56 m
- Maximum net head at 350 $m^3/s$ powerhouse discharge with no flow over the sluiceway: max $H_n$ = 7.97 m
- Minimum net head for operation in the network during flood conditions, corresponding to a river discharge of approximately 3,520 $m^3/s$ min $H_n$ = 2.4 m

A potential increase of the net head up to 0.8 m due to future degradation of the downstream riverbed was taken into account when setting the turbine centre line at 51.40 m asl.

6. Design Flood

During the Feasibility Study, analyses have been undertaken showing that the maximum flood release resulting from the 1:10,000 year inflow into Lake Nasser would not result in a higher release that the 7.000 $m^3/s$ emergency release set by the MOPWWR to achieve drawdown of the HAD reservoir in case of emergency. This is described in Appendix J.

Design floods to be safely evacuated by the sluiceway of the New Barrage will be:

- the 1:10,000 year flood routed through Lake Nasser and released from the HAD at a flow rate of 5,700 $m^3/s$, and
- the 7,000 $m^3/s$ emergency flood releases from the HAD.

For the 1:10,000 year flood, a flood surcharge above the normal operating level (NOL) of the headpond shall not occur. This determines the crest elevation of the sluiceway sill for a given length.

7 Limitation of Reservoir Surcharge During Emergency Flood

It was a requirement introduced by the POE that the flood level during the emergency discharge should not rise above historic flood levels experienced in the years before commissioning of the HAD, especially those levels occurring in 1959 and 1960, years of high floods. The MOPWWR has limited the maximum headpond surcharge at the New Barrage to 67.4 m asl, 1.5 m above NOL. Accordingly the crest of the sluiceway sill must be set at an admissable level.

8 Additional Safety for Evacuation of Emergency Release

The sluiceway with all gates open should have the capacity to evacuate the emergency release of 7,000 $m^3/s$ without participation of the navigation lock. However, the MOPWWR requires that the navigation lock and the upstream and downstream approaches are designed for participation in flood evacuation. This is seen as an additional safety for releasing the emergency discharge. Preparation of the navigation lock for flood evacuation requires a type of gate at the upstream portal which is able to operate under flood conditions.

9. Width and Number of Sluiceway Gates

The POE introduced the requirement that the gate width of the New Barrage sluiceway should be equivalent to that of the navigation lock (17.0 m) to allow interchanging of the stoplogs, if necessary.

Following confirmation of the 7,000 $m^3/s$ emergency release being the maximum discharge to be evacuated by the sluiceway and the associated limitation of the headpond surcharge, the number of gate openings was determined to be seven, see Appendix L1.

10. Diversion Flood Discharge

The diversion flood discharge was confirmed to be 2,900 $m^3/s$. As shown in Appendix L2, this corresponds to a release from the HAD of a flood inflow to Lake Nasser with a recurrence interval of 1:100 years. This constitutes a high safety for the construction stage of the New Barrage.

11. Freeboard Allowance and Crest Level of Main Barrage Structures

The freeboard allowance of the main barrage structures shall allow for:

- Water surface set-up by wind,
- Effect of wave run-up on sloping embankments or wave breaking on vertical walls, and
- Surges as a result of load rejection from the powerplant

with an additional safety margin.

The safety margin for normal operation, including the surges from load rejection, shall be more than 1.5 m, whereas for the safety margin during the emergency release, 1 m is considered acceptable.

From the detailed calculations contained in Appendix L1 and the safety margins, the crest elevation of the main barrage components is 69.00 m asl.

12. Sluiceway Gates

In accordance with recommendations made by the POE, all sluiceway gates shall be equipped with a flap. The release capacity of all flaps together shall be sufficient to release the river discharge exceeding the capacity of the turbines (1,840 $m^3/s$) during the months of peak irrigation demand. This ability to operate the sluiceway avoids the need to lift the main gates during the high flow season or when small differential discharges between powerplant and river flow occur.

13. Dimensions of the Navigation Lock

As for the new navigation locks at the New Esna and Naga Hammadi Barrages, the Gener[illegible]thority of Nile Transport (GANT) requires that the chamber of the navigation lock at the New Barra[illegible] based on the following main dimensions:

| | |
|---|---|
| Net length | 170 m |
| Net width | 17 m |

The level of the downstream sill shall allow for a water depth of 3.0 m below river level at mini[illegible]m flow conditions of 350 $m^3/s$. This value already considers the predicted decrease in downstream w[illegible]r levels due to future riverbed degradation. The adopted floor elevation of the downstream approach and lock chamber is 54.90 m asl.

14. Continued River Navigation

During construction river navigation through the site of the New Barrage shall not be interrupted, but some restrictions to navigation can be imposed during the "closure period" of the irrigation systems supplied by the River Nile. The navigation lock must be passable at unimpounded headwater levels during the construction of the closure dam. For this purpose, the sill of the upstream portal of the navigation lock shall be sufficiently low to guarantee the required water depth (keel clearance of 0.5 m plus draught 1.8 m). This requires that the upstream forebay be excavated to sufficient depth.

Both during construction and after implementation of the project the maximum discharge under which navigation must be possible is 2,900 $m^3/s$. The minimum discharge for navigation is 350 $m^3/s$, but during construction reduced keel clearance or reduced permissible draught of vessels sluicing can be accepted for discharges lower than 700 $m^3/s$.

15. Limitation of the Width of the Diversion Canal

For the period of river diversion during construction of the New Barrage, the GANT has requested a minimum navigation path width of 100 m for safe navigation in the diversion canal, considering that two vessels have to pass each other.

During the Interim Study, the diversion was studied with a wider variant of the canal with limited riprap protection of the banks and the canal bottom, and a narrow variant entirely protected by riprap against erosion. The solution with the least cost and the least environmental impact was the narrow diversion canal, which was adopted in the Feasibility Phase. The shape of the narrow diversion canal was iteratively improved, as described in Appendix L3.

16. Bridge on the New Barrage

The standard version of the Bridge on the New Barrage shall be a service bridge, the size of which depends on operational requirements. The other possible solution shall be a public road bridge either at low level with bascule bridge across the navigation lock or an elevated bridge with a fixed bridge platform across the lock. The clearance between the tailwater level at maximum navigable river flow and the underside of any structure crossing the navigation lock is required to be 13 m. The public road bridge shall be designed to highway standard and for a design load of 60t.

17. Riprap Protection against High Flow Velocities

For the areas which need additional protection against higher flow velocities by riprap, such as those in the vicinity of the structures (intake and tailrace), the thickness of protection and range in diameter of riprap elements were classified for different flow velocities. For this purpose the admissible flow velocities in the riverbed under natural conditions were evaluated by empirical formulae resulting in the riprap specification given in Appendix L3. Riprap protection is limited to some 770 m upstream and some 850 m downstream of the New Barrage, taking into account some scour at larger distances to the main concrete structures.

18. Separation of Public Traffic from Operational Areas on the Barrage

Separation of the main road for public traffic and the area served by the main gantry cranes for operation and maintenance is desirable from the viewpoints of safety and uninterrupted cross-river traffic. As explained in Chapter 4, this requirement can only be met by relocating the power intakes further upstream, and the road bridge on the powerhouse compartments above the draft tubes, at sufficient distance from the runway of the cranes. The main gantry cranes serve the entire length of the powerhouse and sluiceway, except the downstream portal of the navigation lock. The latter is not considered to be necessary for the small maintenance requirement of the downstream lock gates.

19. Surges in the Headpond

Hydropower operation could induce surges in the headpond and the tailwater. Critical surges would result from the case of transmission failure, after which the sluiceway would have to be fully operational within five minutes. During the critical period until the sluiceway assumes the full powerplant discharge, releases from the powerplant would constitute 50% of the total for approximately two minutes. Maximum surge height could reach up to 0.30 m but would attenuate considerably within a few kilometres upstream of the structure.

20. Seepage Cutoff Wall for Construction Pit

From the results of the borehole investigations, it was concluded that the construction pit be enclosed by a deep seepage cutoff ring wall which intersects permeable layers of gravel and sand at depth Connection to the confirmed lower sealing layer of clay should be achieved by keying into the clay layer with a minimum depth of 2m. The seepage cutoff ring wall should be entirely completed before commencement of dewatering of the construction pit below prevailing water level.

21. Height of Cofferdams for Construction Pit

The crest level of the cofferdams for the construction pit should be well above the artesian head encountered during geotechnical investigations in this study. This avoids flow developing through the trench of the cutoff wall from layers under artesian pressure.

22. Safety of Construction Pit against Uplift

The factor of safety against uplift of the dewatered construction pit encased by the cutoff wall and the clay layer shall be at least 1.1. This is discussed in Appendix F.

23. Single-Stage Construction Pit

As a result of the Interim Study comparing single and staged construction pits, it was concluded that under the prevailing site conditions and in view of the considerable depth of the pit for the powerhouse foundation, only the single stage construction pit is viable. A staged construction pit would not only require a considerable extension of the construction period but would also dramatically increase the additional quantities involved in the cofferdam and diaphragm walls.

24. Type of Switchgear at the New Barrage

Comparative studies were made on alternative types of switchyards (see Appendix P), from which the conventional high voltage outdoor switchgear was selected as the least cost solution. The selection of this type of switchgear was co-ordinated with HPPEA/EEA which prefer the conventional type due to maintenance procedures common to their other plant in the system.

25. Transmission Voltage Between the New Barrage and Naga Hammadi Substation

Comparative studies between transmission voltages of 132 and 220 kV for a double circuit transmission line (see Appendix P) indicated a large cost advantage for the 132 kV level at unreduced safety of supply. In view of coming network planning, it was however decided by EEA to select the 220 kV transmission voltage.

26. Control of Barrage and Powerplant Functions

Control and monitoring of the New Barrage project will be divided between two organizations, the EEA being responsible for the operation of the hydropower plant and the MOPWWR being responsible for the total control of releases from the HAD to the downstream reaches of the River Nile, the control of the

headpond level which affects the position of the head regulators at the irrigation canals, and finally the control of the navigation lock.

For the hydropower plant it was requested by EEA that the control room be located in the powerplant. Control data will also be displayed in EEA's administration building.

The operation of the sluiceway gates will be linked to the operation of the hydropower plant for the following two cases:

- During the period when river discharges exceed the powerplant discharges.
- In case of load rejection due to failure of plant or transmission, the sluiceway gates must be opened automatically within some minutes

Direct operation of the sluiceway gates and flaps will be the responsibility of the MOPWWR. The MOPWWR control room will also contain a display of the headpond levels and powerplant discharges (linked to EEA's data acquisition system).

## 3.3 NEW BARRAGE FUNCTIONS AND RELATED HEADPOND LEVELS

From the above criteria and calculations in Appendices L1 and L4, the functions and related headpond levels for the New Barrage over the full range of river discharges considered are presented in Table 3.1

Table 3.1: NEW BARRAGE FUNCTIONS

| River Discharge Conditions | Discharge Rate $m^3/s$ | Barrage Function | Headpond Level m asl |
|---|---|---|---|
| Range of Normal Discharge | 350 - 1,840 | - Irrigation Supply<br>- Hydropower Generation<br>- Sluicing for Navigation<br>- All Sluiceway Gates Closed | 65.90 |
| Range of High Irrigation Demand | 1,840 - 2,400 | - Irrigation Supply<br>- Hydropower Generation<br>- Sluicing for Navigation<br>- Sluiceway Flap Gates Operating | 65.90 |
| Range of High River Discharge (upper limit exceeds $Q_{100}$) | 2,400 - 2,900 | - Irrigation Supply<br>- Hydropower Generation<br>- Sluicing for Navigation (upper limit)<br>- Sluiceway Gates Operating | 65.90 |
| 1:10000 Year Flood Discharge | 5,700 | - Curtailed Irrigation Supply<br>- Hydropower Plant Closed above 3,520 $m^3/s$<br>- Navigation Lock Closed<br>- All Sluiceway Gates Fully Open at 5,700 $m^3/s$<br>- No Headpond Surcharge | 65.90 |
| Emergency Discharge | 7,000 | - Hydropower Plant Closed<br>- Navigation Lock Closed<br>- All Sluiceway Gates Open | 67.15 |

# 4. NEW BARRAGE

## 4.1 LOCATION AND LAYOUT ARRANGEMENT

The location of the New Barrage, as proposed at Conceptual and Interim Study levels, was reconfirmed by the results of the geotechnical investigations described in Section 2.7 and detailed in Appendix E. Based on the results of the borehole investigations, it can be assumed with safety that the future construction pit is completely underlain at depth by a continuous clay layer, with a thickness varying between 6 and 12m. This was the primary reason governing site selection.

The barrage components and their sequence of integration into the New Barrage layout are shown in plan in Album Nos. 7, 9 and 15, and in section perpendicular to the river axis in Album No. 16. The combination of the main concrete structures shown on the drawings is similar to that developed in the Interim Study. With the aim of minimizing interference to navigation resulting from the approach flow to the hydropower station, the sluiceway is located between the hydropower plant and the navigation lock since the sluiceway is expected to be operated for only relatively short periods during the three high-flow months (June to August) with discharges up to some 600 $m^3/s$ or when the powerplant is out of operation.

The sluiceway and the hydropower plant are located within the original course of the river. Aiming to achieve a regular distribution of flow velocities as near downstream of the New Barrage as possible, the powerhouse is located on the left side of the river. It is suggested that this arrangement minimises impacts on river morphology since during the major periods of the year, only the powerhouse operates. Even in the three summer months operating together with the sluiceway, the powerhouse exit flow is more concentrated than the release from the sluiceway. As a result, the navigation lock is located on the right bank, providing for a safe approach to and release from the navigation lock (Album No.7).

Upon submission of the draft feasibility report, it was however requested that a layout with the navigation lock on the left bank shall be added in the Album of Drawings. In this arrangement shown on Album no. 51A, the navigation lock requires extended guidewalls. Although the overall size of the construction pit is only insignificantly larger, the longer diversion canal implies higher cost and consumption of agricultural land.

The three structures are separated by piers, which serve to separate the different floor levels of the approaches to the structures. For the hydropower intake, sluiceway, and navigation locks the approach floor levels are 42.9 m asl (lowest), 49.1 m asl, and 56.6 m asl respectively. All piers are compact reinforced concrete structures, which are partly filled with gravel to increase weight with the internal water level adapting to the river water level.

The navigation lock and the sluiceway are separated by the sluiceway abutment pier with an adjacent connecting structure which is used to accommodate the filling and emptying ducts of the navigation lock (intake is from the headpond and outlet to the tailpond) and the operating and service gates.

The hydropower plant, which requires the lowest foundation level, is separated from the sluiceway by the intermediate pier (Album Nos. 25 and 27) which also has the minimum foundation level of 38.5 m asl. The shape of the upstream end of this pier is important in maintaining adequate approach flow conditions to the powerhouse when operated separately or in combination with the sluiceway. The shape may have to be adjusted during the hydraulic model investigations (Appendix L6).

To the left, the powerplant ends with the abutment pier. This absorbs the earth pressure from earth fill used to form the unloading platform adjacent to the small artificial island remaining from the original west bank of the river after construction of the diversion canal.

The riverbed at this location is narrow and river diversion during construction will require a separa temporary diversion canal on the west bank of the River Nile.

During the Feasibility Study, a number of aspects of the layout evolved further which also influence the design:

- Operationally for maintenance and access with loads to the sluiceway and powerplant, it necessary that 2 gantry cranes are able to travel along the entire length of both structur from the unloading platform between the diversion canal and powerplant. Althoug desirable, it was not considered essential at this stage that the main gantry cranes are able cross the downstream portal of the navigation lock. This aspect is discussed in Sectio 4.10.

- The design of the New Barrage was required to be made for two conditions of access, or with a service bridge on the barrage to be used for operational purposes only, the othe being a public traffic bridge with appropriate road connections. For operational purpose the service bridge can be relatively small and access crossing the navigation lock can be b a bascule bridge. For public traffic, there remain two solutions which depend on th estimated future volume of traffic on the New Barrage, taking into account that the existin Barrage remains operational for cross river traffic. The two alternatives for public traffic o the New Barrage were a low level bridge on the sluiceway extended by a bascule bridg over the navigation lock, or a high level structure with a fixed bridge crossing th navigation lock with appropriate clearance.

- Public traffic must remain uninterrupted during operation and maintenance procedure involving the main gantry cranes. Hence, separation of the public traffic road from th runway of the cranes is required. This condition could be met by locating the crane runwa upstream and the public road downstream above the draft tubes of the powerhouse. Th spacing of the crane beams is then determined by the width of the main powerhouse ha (see Album No. 20). The space between the upstream and downstream stoplogs of th sluiceway is coordinated with the beams of the crane runway.

- The layout with service bridge shall not preclude the later installation of a high level publi road bridge.

The inclusion of these aspects into the layout resulted in a substantial change to the design of th powerhouse from that shown in the Conceptual Study. The new design now requires that the powe intakes be more upstream from the sluiceway structure with the intermediate pier between bot project components enlarged. Provisions for later construction of the high level bridge required tha the compartmentalization of the space above the draft tubes meets the requirements for the high leve bridge.

For hydraulic reasons, the downstream stoplogs shall be located at the end of the draft tubes, whic requires a downstream service crane beam bridge separate from the public road bridge.

The navigation lock is located on the left side of the sluiceway, with the upstream and downstrean approaches cut into the banks of El Dom Island. The location of the downstream portal is as near a possible to the road bridge, with the aim of reducing construction quantities and costs. For the sam reason, the crane beams are placed on a level a short distance to the water surface, which does no permit the main gantry crane to serve the downstream portal of the navigation lock.

For the high level road crossing of the navigation lock, a clearance of 13 m above maximun navigable water level at 2,900 $m^3/s$ is required. Road crossing of the navigation lock is only viabl downstream of the downstream portal. The road must be elevated by ramps on both sides of the mai

concrete structures if by architectural reasons a ramp on the concrete structure (eg on the sluiceway) is not permitted.

A summary of the principal features of the components of the New Barrage are presented in Tables 4.2 to 4.7 at the end of this Chapter.

## 4.2 CONSTRUCTION PIT

Prior to construction of the main concrete structures, the construction pit must be excavated. This will extend across the entire width of the riverbed plus limited portions of both river banks. From the detailed studies made in the Interim Phase, a single-stage construction pit was selected for planning of the final layout of the New Barrage, around which the diversion canal is excavated. The extension of the construction pit onto El Dom Island side is determined by the length of the navigation lock, including the upstream concrete guide walls.

Album No. 10 shows the arrangement of the construction pit with the diversion canal while a more detailed plan of the pit is presented in Album No. 11.

The lateral extent of the construction pit is determined by the allowable excavation slopes during construction, which are in turn based on the properties of the surrounding sand. With embankment slopes of 1V:3H and an excavation depth of some 25 m below existing ground levels, the total width of the construction pit is some 450 m while the length at the side of the navigation lock extends to around 600 m. The total quantities to be excavated depend largely on the depth and extent of the existing river channel but are estimated to be approximately 1.53 million $m^3$.

The upstream and downstream boundaries of the construction pit are formed by cofferdams up to 14 m in height. When the diversion canal is completed, construction of the cofferdam will commence with end-dumping of rockfill from barges or the riverbanks, followed by filter material and sandfill. Drawing No. 13 indicates the order of placement with the second stage sandfill being on the upstream side of the first stage rockfill. This will ensure that sandy material penetrates the rockfill and supports the second stage sand embankment.

A diaphragm wall is required to ensure adequate safety when dewatering. Layers of sand and gravel of high permeability, which can be expected to transmit a significant amount of seepage under artesian conditions, would otherwise make this task difficult. Conditions will nonetheless be less severe than immediately downstream of the existing Barrage. The platform for the preparation of the diaphragm wall must, however, remain well above the hydraulic grade line of artesian pressure which extends from the headpond level downstream.

The method of establishing the diaphragm wall will be by slurry trenching rather than bored piles. Although some risks remain, this approach is recommended based on lower artesian pressures observed in this area, the fact that no structures exist in the near vicinity, and the comparatively higher progress rate achievable by trenching during construction.

The diaphragm wall shall be keyed some 2 m into the clay layer which was encountered during drilling at an average elevation of 15 m asl. Based on the grid of boreholes drilled during both the Interim and Feasibility Stages, it is assumed with safety that this layer underlies the entire area of the construction pit.

The diaphragm wall will have a constant thickness of 1.0 m and will penetrate both cofferdam and riverbed material. It will enclose the entire construction pit. The diaphragm wall section along the right bank will be taken as part of the permanent works, described in Section 4.8. In view of the proposed seepage cutoff to the construction pit, the estimated inflow will depend largely on the quality of the diaphragm wall but is expected to be minimal.

With the tight encasing of the construction pit, uplift under the clay layer can be critical when excavating and dewatering the construction pit to the lowest foundation level. The calculations given in Appendix F indicate that sufficient factors of safety, around 1.2, can be maintained at the minimum foundation level required for the powerhouse and foundation piers, which is 38.5 m asl, some 24.0 m below the water level during passage of the diversion flood.

By comparing Album Nos. 10 and 17, those areas up- and downstream of the powerhouse and sluiceway over which filter and riprap protection are required can be identified.

Access to different levels of the construction pit during construction is provided by a system of ramps on the inner slopes of the excavations, accessible from the side of El Dom Island.

## 4.3 DIVERSION CANAL

Minimizing the excavation quantities for the diversion canal requires that the canal curves around the shorter side of the pit where the powerplant is located constituting as well the preferred alignment for hydraulic reasons.

The diversion canal extends to the west side of the construction pit and has an approximate length of some 1,100 m and a base (sole) at 52.0 m asl. During the minimum navigable river discharge of 350 $m^3/s$, the flow depth will be 5.9 m.

The minimum width at the sole is 125/135 m, or 25 m wider than required by the GANT. The width of the canal was not, however, further decreased. Average flow velocities admissible during the four years of construction may increase by some 20% above maximum average flow velocities under natural conditions in the River Nile, which in this river section are 1.3 m/s. In view of the temporary nature of the diversion canal, its width should be minimized to reduce cost.

Other criteria considered in designing the diversion canal were:

- Velocities transverse to the navigation route must not exceed 0.3 m/s.
- Navigation shall not be impaired for all river discharges up to 2,400 $m^3/s$, and still be possible for discharges up to 2,900 $m^3/s$ through the diversion canal. Discharges exceeding 2,000 $m^3/s$ occur over only 3 months of the year from June to August.
- Erosion criteria limit maximum local velocities in the riprap-lined canal to about 2.4 m/s for riprap type III (Appendix L3, Section 3.6).

Maximum velocities occurring during operation of the diversion canal depend on the canal alignment and the shape of the cofferdam around the construction pit. Upstream, the velocity distribution depends on the velocity profile of the river flow at the canal inlet. Downstream, the velocity distribution is affected by the transition to the natural river cross-section.

With the aim of visualizing the local velocities in the diversion canal and of ensuring suitable flow conditions, both for navigation and for the stability of the canal against local erosion, mathematical modelling of the flow pattern for different diversion canal arrangements was undertaken. Modelling was confined to conditions when the cofferdams are closed and flow is solely through the diversion canal.

Seven alternative shapes described in Appendix L3, were analyzed for different bed widths of the canal, inlet and outlet conditions, and alignment of the right bank of the diversion canal towards the construction pit. From the seven shapes, the associated flow patterns for versions 5, 6, and 7 (Figure 4-1) are shown in Album Nos. 67 to 71. The selected solution (Shape 6) provides a sufficiently wide navigable fairway within a canal of 125 m minimum bottom width. Average velocities in the 100 m

wide fairway of less than 1.5 m/s are anticipated for a discharge of 2,400 $m^3/s$ with maximum local velocities of 1.65 m/s. For 2,900 $m^3/s$, maximum velocities are estimated to be 1.9 m/s. For the adopted canal shape, the extent of flow separation zones was reduced to a minimum. Flow separation naturally occurs in areas near the riverbanks with flow deceleration, for example near the contours of the construction pit at the inlet and outlet.

River navigation upstream would follow a course close to the left bank and would change to the right bank only at the downstream end of the diversion canal.

## 4.4 SLUICEWAY

The sluiceway structure, shown on Album Nos. 9, 10, 13, 24 and 25, was dimensioned for a capacity of 5,700 $m^3/s$ with full opening of all gates and maintaining the headpond level at 65.9 m asl. This discharge corresponds to the release from the HAD of an inflow flood to Lake Nasser with a 1:10,000 year recurrence interval. The evacuation of floods resulting from natural flood inflow into Lake Nasser is detailed in Appendix J, Table J7.

The 1:10,000 year event is regarded as a very conservative design flood for the New Barrage. However, there is a requirement imposed by the MOPWWR to dimension the barrages in Upper Egypt for an emergency release of 7,000 $m^3/s$ from the HAD. As discussed in Appendix J, the HAD is able to discharge at flows considerably higher than this, but it can be assumed that even during an emergency, releases will not exceed this upper value. Releases of this magnitude may also be required under certain conditions to draw the reservoir level down to allow maintenance works on the power intakes of the HAD.

For the emergency release, it was requested by the MOPWWR that the New Barrage could evacuate the 7,000 $m^3/s$ by the sluiceway only, with full opening of all gates, and the headpond level rise limited to 67.4 m asl.

A key parameter in the hydraulic design of the sluiceway and downstream apron is the tailwater rating curve given in Figure 2-4.

The sluiceway of the New Barrage consists of 7 bays, each equipped with a 17 m wide radial gate. On top of each of the radial gates, flap gates are arranged for discharging water in excess of the turbine discharge capacity (occurring in the months of June to August), for fine regulation of the headpond level, and for the flushing of floating weed should its removal not be possible.

The hydraulic design of the sluiceway of the New Barrage is similar to that of the existing sluiceway at the New Esna Barrage. However, the sill and apron levels and the length of the apron have been adapted to the prevailing tailwater conditions at the New Naga Hammadi Barrage site.

Due to the depth of the sluiceway sill, the outflow beneath the radial gates will be submerged for the entire range of discharges. The typical ogee form for spillway crests is, therefore, not applicable. In the case of very high floods, the gates would be fully lifted and the flow conditions would remain subcritical. As usual in such cases, final dimensions of the structure must be confirmed by hydraulic model tests, which presently are underway (see Appendix L6).

The gates of the sluiceway structure are separated by 4m wide piers. The block joint system of the civil structure of the sill provides for an expansion joint in the middle of each opening, thus sectioning the structural system into 6 blocks of 21m width, each with a central pier, and two end blocks including the dividing piers at each end of the sluiceway. On the powerplant side, the end block of the sluiceway includes the intermediate pier, except its downstream portion which is separated by another joint. On the side of the navigation lock, the end block includes the sluiceway abutment pier, except at the downstream end. The block system was chosen with the expectation of sufficient density of the foundation soil at 45.3 m asl and the positive experience at the New Esna

Barrage where negligible differential settlements have occurred. Details of the foundation conditions are described in Appendix F.

A section along the axis of a sluiceway opening is shown on Album No. 35, extending over the range of the piers and the downstream apron. The operating range of the gate and flap is restricted to the area between the runway beams of the main gantry cranes, including the upstream stoplogs. The slots for the downstream stoplogs are on the downstream side of the gantry crane runway beam.

The service bridge is located near the downstream stoplogs. The space downstream on the piers can accommodate the elevation road bridge (see Album No. 46).

Each of the gates with a clear width of 17.0 m and a height of 13.5 m, including the flaps, is moved by two oil-operated hydraulic hoists located in the piers. It is a safety requirement that the hydraulic hoists be capable of holding the gate with one servomotor only. The gates are capable of closing the sluices under action of gravity for any conditions of head and discharge.

All gates are equipped with an upper flap gate for regulating smaller differential releases between the powerplant and sluiceway and for release of floating debris and weeds. The flaps together with the turbines of the hydropower plant will enable release of the full river discharge during the high flow (high irrigation demand) season. In case of sudden load rejection by the powerplant, the sluiceway gates will be opened to assume the discharge at which the powerplant was operating.

The stoplog elements are interchangeable with those from the navigation lock and are stored in the dogging device above each sluiceway opening. The elements can be raised and lowered by the main gantry crane under balanced head conditions only.

Album Nos. 17 and 18 show the entire length of the approach to and from the sluiceway sill, which is covered by riprap on top of a filter layer. Riprap extends some 430 m upstream and some 850 m downstream from the sill.

## 4.5 HYDROPOWER PLANT

### 4.5.1 Operating Conditions

All river discharges will normally pass through the hydropower plant except during the high flow period during the months of June to August. With the units operating, the maximum discharge capacity of the hydropower plant will be 1,840 $m^3/s$. Maximum generating capacity of the generators is 64 MW (4 x 16 MW) at prevailing head conditions, which cover a net head range of 3.73 m to 7.97 m. The highest head corresponds to a headpond level of 65.9 m asl and a tailwater level of 57.86 m asl (based on a discharge of 350 $m^3/s$). The lowest head corresponds to maximum normal river flow during the high-flow season of 2,320 $m^3/s$. During floods, the hydropower plant will operate in combination with the sluiceway up to a maximum total river discharge of 3,520 $m^3/s$, which corresponds to a minimum head of 2.40m. For higher river discharges, the powerhouse would be closed and would not be able to participate in flood release. Discharges at very low head through the powerplant result in uncontrolled operation of the turbines; in any case the discharge at these heads would not be significant.

The most critical case for turbine operation is the full-load rejection, eg by transmission loss or similar failure of the powerplant. If this occurred the distributor of the turbines would have to close, which would result in a high-speed rise of the turbine runner and surges in the headpond and the tailwater of the New Barrage. The sailing operation of the units is therefore applied, which limits the overspeed and maintains about 50% of the discharge through the powerhouse for some 2 minutes. During this time, the sluiceway gates would be activated and within the following 3 minutes opened to convey the former powerplant discharge. Details of this operation are given in Appendix O.

The dimensioning of the generating units is described in Appendix O. The low setting of the axis of the bulb turbines at 51.4 m asl, that is 6.5 m below minimum tailwater level, already takes into consideration the reduced tailwater elevation which is estimated to be established within 50 years by riverbed degradation.

### 4.5.2 Powerhouse Design

As shown in Album No. 19, the powerhouse consists of 4 unit bays, each two being combined into one structural unit block of the civil structure. The length of the block is determined by adding the dimensions of the structures in which the following single components (see Album No. 20) are contained:

(i) trash rack with raking machine,
(ii) upstream stoplogs for dewatering of the turbine pit,
(iii) generator-turbine bulb unit with foundation, vertical shaft bearing with integrated access shafts and subsequent turbine encasing with distributor blades and turbine runner, and
(iv) draft tube with the transition of the discharge section to the outlet square shape, and tailwater gates at the end of the daft tube.

The total length required for all standard components which determine the total length of the powerhouse unit blocks, is 72.0 m. The 14.2 m width of a unit block results from the required intake area and the setting of the turbine axis.

The powerplant design contains a main powerhouse hall, which extends over the total length of the generator-turbine sets including the intermediate and abutment piers, and which is closed against the exterior by removable roof covers above each unit and both piers. The design avoids the large open pit above the generator bulb with any associated tendency for hydraulic surges. The main powerhouse hall is served by two interior cranes for a maximum load of 10 t each, sufficient for the loads to be moved for maintenance of mechanical and electrical equipment. For extraordinary cases of repair requiring larger load capacity, the roof cover above single generating units can be removed and the main gantry cranes used.

In the options with a public traffic road, the traffic is well separated from all operations concerning the powerhouse, being located a distance of about 6.4 m downstream from the runway of the main gantry cranes. The space below the road and above the draft tubes is used to accommodate all electrical and auxiliary equipment including the main transformers. Space in both side piers is integrated into the powerhouse compartmentalisation.

The outline design of the civil structure is shown in sections on Album Nos. 20, 24 and 25 and in plan for different levels on Album Nos. 19, 21, 22 and 23. The overall dimensions of one structural block with two units is 35.4 m in width and 72.0 m in length, with a total height between lowest foundation level and road platform of 30.4 m. The joints between the blocks are located in the piers and for the end blocks at the side of the piers.

The principles on which the design is developed are:

(i) In order to obtain sufficient trashrack area at limited depth and width, the trashrack is inclined. The gross horizontal velocity at the design discharge of 1,280 $m^3/s$ is 1.12 m/s. The trashrack area cannot be closed upstream. A trashrack cleaning machine is provided to remove trash and waterweeds arriving from upstream. Trash will be accumulated in a transverse canal and removed.

(ii) The area of the generator bulb and turbine is closed against the flow and accessible through the main powerhouse hall, which covers the area above the steel-lined components of the turbine canal and draft tube. The 7.4 m diameter bulb and turbine are only vertically

supported within the arrangement of one large steel-lined column with internal shaft. Within this supporting structure, there is permanent access to the interior of the bulb generator and to the gallery underneath the steel casing of the turbine runner and distributor blades. Access through the turbine shaft into the turbine canal is only possible when the upstream and downstream emergency gates are closed and the water from the turbine canal has been evacuated. For access, the cover of the steel casing has to be dismantled. The generator shaft within the supporting structure contains all cables for connection to the medium-voltage switchgear.

(iii) The guide bearing of the runner and generator is accessible via the gallery running underneath each of the units at an elevation of 43.5 m asl. This gallery is connected within the supporting structure to the main powerhouse floor at 61.25 m asl by a shaft, through which loads can be moved by the powerhouse cranes. The gallery connects all unit blocks with the access shaft in the left abutment pier and with the staircase and elevator shaft.

(iv) Dewatering of the turbine canal and associated draft tube is possible by lowering the emergency gates upstream in front of the generator bulb and downstream at the end of the draft tube. The upstream gates can be closed under any flow condition. The gates are stored in the upper part of the gate slot and can be lowered by the main gantry crane at any time. The downstream stoplogs are set for erection, inspection and maintenance of the units by a crane from a separate service bridge. Stoplogs are stored above the draft tube outlets. The turbine canal and draft tube can be totally dewatered by pumping from the dewatering sump contained in the intermediate pier between powerplant and sluiceway, which connects by gallery to each of the unit blocks.

(v) All electrical equipment is contained in the rooms above the draft tubes and - if provided - underneath the public road, accessible from the main powerhouse hall and the staircase and elevator shaft in each end pier. Separate compartments are provided in each unit block near the units for the following groups of equipment: main transformers, 11 kV cubicles and auxiliary transformers, and the excitation and voltage-regulating cubicles. Other equipment common for all units are contained in the compartments on the downstream wall of the powerhouse above the drafttube. The cable duct from the abutment pier leading to the conventional switchyard is indicated in Album No. 19.

4.5.3 Electrical Equipment and Connection of the Powerplant to the Unified Power System

All electrical equipment is contained in the powerhouse at floor levels 61.25, 60.50 and 64.50 m asl. 220 kV dry cables extend to the conventional switchyard, on the west (left) bank, through a cable duct. From there, the 220 kV circuit transmission overhead line follows a 23 km route parallel to the existing 66 kV line to the Naga Hammadi substation, which serves the existing Barrage, see Album No. 32.

The single-line diagram is shown on Album No. 29. Two main transformers with two generators connected to each were selected, this being the least-cost solution. The main transformers step up the 11 kV generator voltage to 220 kV and feed into twin busbars. Between each generator and its transformer, the generator output switchgear is housed in an 11 kV cubicle. Power is evacuated from the transformer terminals via the above-mentioned 220 kV dry cable connection to the conventional outdoor switchgear.

The service station auxiliaries include two main service station cubicles, each fed by an 11,000/400 V step-down transformer connected to a generator terminal and an emergency service station cubicle.

The 18.8 MVA generators with a nominal speed of 73.2 rpm are directly coupled to the turbines and contained in the bulb. Cooling is provided by air-with-water heat exchangers. To accommodate the future increased power output resulting from the increased generating head by river degradation, the

specified insulation is of Class F (100 °C) with a maximum temperature rise under present conditions corresponding to Class B (80 °C).

The power factor for operation under present tailwater conditions with a maximum generator output of 16 MW is 0.85. Calculations of the future power increase due to the ongoing river degradation show that the maximum power output within 30 years of operation (2036) could increase to approximately 17.8 MW and within 50 years (2056) to approximately 18.5 MW. For the possible power increase during the economic life of the project, the higher heat emission of the generators will remain within the limits for a Class F insulation if the power factor is limited accordingly (year 2036: $\cos \varphi_{min} = 0.90$). EEA has no objection against correcting the power factor in the future to accommodate the possible increase of capacity of the NHB powerplant.

Details on the generator output switchgear, the powerplant auxiliaries and the main transformers are contained in Appendix P.

The high-voltage switchgear located on the west bank adjacent to the backfilled diversion canal will be a conventional outdoor type with double busbars. The switchgear includes 2 incoming bays from the main transformers, two outgoing bays to the transmission line and one bus-coupling bay. The area required for the switchyard would be overall 60 x 50 m, the layout being of the classical low-rise type. The transformer bays of the switchyard will be connected to the main transformer terminals by 132 kV single phase dry cables. The length of the cable connection will be some 420 m. The cables would be laid in a concrete lined trench along the crest of the closure dam. The typical switchyard layout and sections are shown on Album Nos. 30 and 31. The outgoing line bays will be provided with line traps and capacitor-type voltage transformers, apt for coupling with a power-line-carrier communication system.

The line connects to the Unified Power System (UPS) via the Naga Hammadi substation located approximately 23 km from the New Barrage. This substation is one of the largest in Egypt and includes all voltage levels from 11 kV to 500 kV, see Album No. 33.

## 4.6 NAVIGATION LOCK

The navigation lock is constructed on the right bank of the river formed by El Dom Island. The following explanations refer to the layout and sections given in Album Nos. 34, 37 and 38.

The approach on the upstream side of the lock requires relatively large excavations down to about 54.0 m asl to allow passage of river traffic during construction when the headpond is still not impounded but the diversion canal is closed and the river flow passes the sluiceway. The low upstream sole (base) of the approaches to the navigation lock at 56.6 m asl is also required for enabling the navigation lock to participate in the evacuation of the emergency release from the HAD, as required by the MOPWWR for additional safety.

The GANT requires a minimum of 3.0 m depth of water in the navigation lock and above the downstream sill to make adequate allowance for future riverbed degradation. The maximum draught of ships navigating the River Nile is 1.8 m and a keel clearance of 0.5 m must be provided. For the minimum navigable river discharge of 350 $m^3/s$, the water level based on the tailwater rating curve in Appendix K is 57.9 m asl. Allowing for the anticipated tailwater decrease within 50 years (0.8m), a water depth of 2.2 m would still remain.

For the period of construction of the closure dam, a total depth of water of 2.30 m is provided to allow for the maximum draught of 1.8 m plus 0.5 m keel clearance at a river discharge of 710 $m^3/s$. For the few weeks of the closure period when the river discharge is reduced to 350 $m^3/s$, the maximum permissible draught will be somewhat limited.

The navigation lock is a 217 m long structure with an L-shaped reinforced concrete cross-section. in which the retaining walls of the chamber use the weight of backfill and water to increase stability. The dimensions are shown on Album Nos. 37 and 38. Seven of the 8 blocks between the portal blocks are 20 m long and of almost similar cross-section. The most downstream section contains the chamber for dissipation and distribution of the filling discharges and is also used for emptying.

The upstream portal section contains the vertical-lift sleeve gate, shown on Figure 4-2, which enables both leaves of the gate to be operated for flood discharge when river navigation is stopped. Similar operation of its lower leaf applies for the short period during construction of the closure dam. Under normal headpond and river-flow conditions, only the upper leaf of the gate is moved by hydraulic hoists, lowering the gate after ships enter. Details of the vertical-lift-sleeve gate and its operation are described in Appendix S. This type of gate has been successfully used for 50 years on the Danube River navigation locks.

The downstream portal contains a hydraulically-driven mitre gate. For accelerated leveling when the water in the lock chamber approaches tailwater level, the two gate leaves each contain a bypass roller gate.

To the left of the filling chamber section, there is the connecting structure between the navigation lock and sluiceway abutment pier. The connecting structure contains the system of ducts for filling and emptying the navigation lock. As can be seen on Album Nos. 34, 36 and 38, the filling duct has its intake within the upstream bay formed by the connecting structure, whereas the outlet of the emptying duct is located within the downstream part of the abutment pier of the sluiceway. Within the navigation lock, both inlet and outlet chambers are located in the sidewall of the distribution and dissipation chamber. The connecting structure contains 2 pairs of gates for each duct for operation and emergency.

The length and width of the upstream and downstream approaches to the navigation lock comply with the navigational requirements as described in Appendix L4. As shown in Album Nos. 72 to 75 and 78 to 81 the cross-current velocities at all navigable discharges both up- and downstream do not exceed 0.3 m/s. In the vicinity of the barrage, higher cross-currents are prevented by guide walls. In the upstream area this results in massive structures due to the low level of the approach.

Filling and emptying times of the navigation lock were calculated in Appendix L4. Both these periods, not exceeding 11 minutes, are not critical for the hydraulic forces acting on ships inside the lock chamber.

## 4.7 CLOSURE DAM AND SIDE DYKES

The closure dam is constructed when the sluiceway structure is operable and the cofferdams are removed to pass the river flow through the sluiceway for the period during wet testing of the generating units. The closure embankment continues in the axis of the future public road on the main concrete structures across the 200 m wide (125 to 135 m at base) diversion canal.

The embankment section shown on Album No. 31 comprises upstream and downstream rockfill toes, which are end-dumped from the riverbanks or from barges. An underlying layer of gravel is provided as a filter to prevent migration of sand from the river bed into the rockfill. The rockfill on the upstream side of the embankment (headwater side) will be of small grain size to prevent a permanent flushing of sand into the rockfill. Under the protection of the two rockfill embankments, the closure dam will then be completed by end-dumping sand from the left-bank deposit. The slopes of the rockfill toes will be at 1V:2H and dumped to elevation 62.0 m asl to allow subsequent works to be completed under dry conditions for river discharges up to some 2,500 $m^3$/s (high irrigation demand season).

Generally, no compaction under water will be performed during construction. When the dumping height is above the water level, a dynamic roller will be used, resulting in very small expected long-term settlements. A sand filter and finally riprap protection will preserve the slopes of the closure dam against erosion.

As for the cofferdams, the slope stability of the closure dam will be well above the required factor of safety against slope failure of 1.3. For potential slip surfaces passing through both sand and rockfill, the location of the more shear-resistant rockfill at the toe of such slip surfaces and the berm width of 7.5 m will favourably influence the slope stability.

The embankment has to be sealed by a diaphragm wall. It will be a slurry trench cutoff wall, similar to that around the cutoff trench but with a width of 0.6 m. The top of the wall is at 67.5 m asl. At this elevation, the closure dam is 15 m wide, which is sufficient to place the equipment for cutoff wall construction.

The seepage exit gradient at the downstream toe of the embankment is acceptable, being less than the critical gradient for fine and medium sand.

There are protection dykes on the left bank, a typical section being shown on the left side of Album No. 14. The protection dyke at the left bank has an average height of only some 2 m, although this varies according to the natural surface. The foundation level is primarily well above the NOL. A separate sealing is therefore not necessary, nor a filter at the downstream toe.

The protection dykes on El Dom Island and the right bank closure dyke in the old river flood channel are similar (see right and bottom of Album No. 14). A permanent impoundment will exist only in front of the backfill. According to the evaluation of the static cone tests (Appendix E), there is no natural sealing within an acceptable distance in the entire area. Nevertheless, no diaphragm wall is required as the natural seepage through the backfill of the floodway will be acceptably low. At the downstream toe of the backfill and the island, a filter will be placed to intercept any seepage water, which will then be led into the tailwater by drainage.

## 4.8 SEEPAGE CONDITIONS AND SAFETY AGAINST UPLIFT

The foundation levels of the powerhouse at 38.6 m asl and of the sluiceway at 45.3 m asl are within or above a layer of gravel and cobbles (Figure 2-8). This layer, with a permeability of $2 * 10^{-3}$ m/s derived from evaluating the pumping test (Appendix E), is decisive for seepage and uplift. Hence, a short seepage cutoff wall is arranged below the upstream ends of the sluiceway and powerhouse unit blocks to intersect that layer. It is not considered necessary that the diaphragm wall penetrates into the clay layer, which has an upper boundary between elevations 13 m asl and 16 m asl, since the reduction of seepage to tolerable limits is achievable with the foreseen depths of the cutoff wall of 11 m and 15 m respectively (28 and 33 m asl).

The diaphragm wall will be a slurry trench cutoff wall as for the temporary works. The thickness will, however, be reduced to 0.6 m since the depth of this cutoff wall will be only 15m or less.

The uplift calculations in Appendix F show that the powerhouse is sufficiently safe against uplift when both turbine canals and draft tubes contained in one unit block are dewatered and the equipment is removed. This considers the critical case with the upstream water level at NOL, and the downstream water level at 62.5 m asl corresponding to a discharge of 2,900 $m^3$/s. For the sluiceway, the downstream apron is the most critical section concerning uplift. The calculations indicate sufficient safety against uplift for the sluiceway also considering long-term riverbed degradation.

To ensure the area of the inlet and outlet structures of the navigation lock is safe against uplift for the case of emptying during maintenance, the diaphragm wall will continue from the sluiceway to the navigation lock and offset in plan to run upstream of the filling block of the navigation lock, as

shown on Album Drawing No. 15. Beyond the navigation lock on the right bank, it will be connected to the diaphragm wall of the temporary works for the construction pit in order to considerably lengthen the seepage path around the navigation lock.

The diaphragm wall on the left side will also be offset in plan under the abutment pier of the connection to the diaphragm wall of the closure dam. Both diaphragm walls - of the powerhouse abutment and of the closure dam, connect to the remainder of the larger diaphragm wall of the temporary works which runs parallel to the backfilled diversion canal.

The downstream ends of the sluiceway apron and draft tubes including the sides of the piers are protected against scouring by a sheet pile wall. The exit gradients of seepage flow with the designed lengths of the cutoff walls and sheet piles under the powerhouse and sluiceway have been estimated by two-dimensional seepage calculations (Figure 4-2), as documented in Appendix F. They are tolerable with a sufficient factor of safety, even for the case that riprap and filter protection would partially fail.

## 4.9 ASSESSMENT OF FLOW CONDITIONS AND RIPRAP PROTECTION REQUIREMENTS

For the purpose of a first assessment of the flow velocity field up- and downstream of the New Barrage without the costly and time-consuming construction and operation of hydraulic model investigations, a two-dimensional mathematical model was applied. The simulations considered discharges ranging from 1,840 $m^3/s$, the maximum turbine discharge, to the emergency release of 7,000 $m^3/s$.

Results of the mathematical model were used for:

- Confirmation of the layouts, orientation of barrage axis, and relative location of the components to the centreline of the river.

- Visualization of the approach flow to the powerhouse and sluiceway under different rates of river flow and powerhouse discharges.

- Visualization of the expansion of the jet issuing from the sluiceway of the barrage, which should produce normal velocity profiles as soon as possible downstream of the barrage, and mitigation of unduly high concentrations of the flow immediately upstream of the barrage.

- Visualization of the spreading of the powerhouse discharges downstream and return of the release to normal river flow patterns.

- Determination of the extent and the class of riprap protection upstream and downstream of the structures to prevent erosion.

- Visualization of possible flow velocities which might affect river navigation in the upstream and downstream approaches to the navigation lock, for the purpose of dimensioning the extension of guidewalls.

In general, the flow patterns obtained from the numerical simulation of the various operation modes confirms the suitability of the New Barrage design. However, the results indicate the need for further optimization of the approach and exit flow conditions to and from the powerhouse by physical modelling.

The most favourable flow conditions in the upstream river reach result from the operation of the powerhouse only, which will be the most frequent operating mode. Simultaneous operation of

powerhouse and sluiceway distorts the flow field, and remedial measures are to be studied in the physical model.

Streamlines are considerably constricted in the immediate vicinity of the barrage, sluiceway and powerhouse, resulting in zones of downstream eddying and a considerable lateral flow component. The flow towards the powerhouse has a lateral velocity component in the case of simultaneous operation of the sluiceway and powerhouse as shown in Album Nos. 73 and 75. Optimization of the layout in this respect is required in the physical model study through the testing of various guide structures.

Flow conditions for vessels approaching the upstream and downstream forebays of the navigation lock are rather good; they should not require further modification. This, however, should also be reconfirmed in the hydraulic model study.

The type and extension of upstream riprap protection is designed according to the permissible flow velocities for the respective riprap types (see Appendix L3, Section 3.6). Riprap is built on filter layers which for the largest sizes Type I and II would be double filters. The distribution of riprap of different size is given on Album No. 17. The load case which is decisive for the heavy protection of the permanent works upstream and downstream is the emergency flood. This applies also to the navigation lock approaches, as it is required that the lock can participate in the release of an emergency flood from HAD, providing additional safety for flood evacuation.

The river flow patterns based on the mathematical modelling indicate riprap protection is required to extend in excess of 850 m downstream of the structure. However, the extent of the riprap has been limited to this length as limited scouring at this distance is not regarded to be of significance to the structure. However, this will have to be investigated in sufficient detail by physical model studies extending further downstream in the river.

Details of the dimensioning of the riprap protection are given in Appendix L3.

The flow conditions in the diversion canal require only that riprap Type IV be used as the flow velocities do not exceed the permissible velocity for this riprap of 2.0 m/s. For the slopes of the protection dykes and upstream cofferdam facing the river, riprap Type III protection is sufficient.

## 4.10 ACCESS AND APPURTENANTS

Three variants of the New Barrage project were studied simultaneously to the assessment of future cross river traffic (the latter described in Appendix V). During the course of the study and independent from the cross river traffic evaluation, it was agreed with the MOPWWR to design the structure for:

(i) a service bridge, for operational purposes only,
(ii) a public road bridge level with the crest of the concrete structures, and
(iii) a public road bridge elevated to a level required to cross the navigation lock with a fixed bridge.

As mentioned in Section 4.1, the service bridge constitutes the basic variant for the New Barrage. The service bridge variant is taken as the basis for the economic evaluation of hydropower and its financial analysis, as the public traffic function of the barrage structure is seen as a separate issue. The service bridge does not preclude the later installation of the high level public road bridge. However, it is recommended that the MOPWWR selects between the variants (i) service bridge and (ii) low level public road bridge before entering into the tender design stage. The total time for construction of the New Barrage will be independent from the bridge variant adopted by the MOPWWR, as the bridge and its road connections are not on the critical path.

The service bridge is part of the basic layout shown on Album Nos 7. 9. 15. 16. 34. 35. 36. and 37. The access to the New Barrage for operational purposes would be from the west side of the existing Barrage, see Album Nos. 3 and 7, via a paved road which runs on top of the left bank protection dyke. A typical section of the permanent access road is shown on Album No. 14. On the backfill of the upstream section of the diversion canal, the road follows the new shore line to the paved unloading area on which the crane rails end. On the powerhouse. the total concrete platform is available for access. Between the powerhouse and the navigation lock filling structure, the sluiceway is spanned by the service bridge, designed for 60 t capacity and with a total width of 12 m.

From the right bank platform near the navigation lock, there is access by a bascule bridge across the downstream portal block of the lock. The size of the bascule bridge is 2 m smaller than for the low level public road variant, but its capacity is sufficient to provide access for heavy loads, eg a mobile crane, from the platform on the right bank to the platform on the lock filling structure where the main gantry crane rails end.

From the secondary road on the right side of El Dom floodway, access for service is by an unpaved road on top of the floodway closure dyke and on the Dom Island protection dyke, see Album Nos. 7 and 14.

The public road alignment of both other project variants will principally follow the course of the protection dykes on the right side of the New Barrage. On the left (west) bank, the road alignment will follow the crest of the closure dam and then for some 1,100 m along the left bank protection dyke from which it branches off to the national road as shown on Album Nos. 39 and 48. The paved service road between the existing Barrage and the junction with the new national road will be maintained for both public traffic bridge variants.

With the low level public road, the sluiceway structure will be spanned by a 143 m wide road bridge of 60 t capacity, level with the concrete structures as shown on Album Nos. 49 to 51. A bascule bridge would be required to cross the navigation lock. As shown by the estimation of future traffic volumes in Appendix V, it can be expected that the low level traffic bridge together with the existing bridge facilities on the existing Barrage would adequately meet predicted long-term cross river traffic in the area.

The difference between the capacities of a high and low level bridge appear to be only marginal. Barges can pass the low level bridge without operating the bascule bridge, which would only be necessary for the higher tourist boats. If the MOPWWR considers it necessary to guarantee uninterrupted flow of traffic crossing the navigation lock, the high level public road would be needed, for which a clearance of 13 m above maximum navigable tailwater level (62.5 m asl at 2,900 $m^3/s$) is required by the GANT. Album Nos. 39 to 47 show the project with the high level public road crossing, some 8 m above the crest of the New Barrage. The elevated bridge necessitates constructing ramps on both sides of the main concrete structures, as indicated in the views on Drawing No. 42. The ramps on both sides would continue by concrete structures with slab foundation on backfill. On the powerhouse side, the road continues on a concrete structure which decreases in height to the level of the closure dam as shown on Album No. 47. The concrete ramp is needed as an embankment ramp would require a wider closure dam which would substantially increase the quantities involved in the structures on the left bank.

It was recommended by the POE that the high level bridge is combined with adequate architectural features, for which an indication is given on Album No. 42. The runway of the two main gantry cranes remains on the low level throughout the length of the concrete structures. The crane beams end on the platform of the intermediate structure between navigation lock and sluiceway, and as a result the gantry cranes cannot travel to the right bank.

On the right side, the high level road bridge requires that the concrete ramp is continued by a rockfill ramp. This requires more space on El Dom Island than would be required for the low level public traffic road.

Appurtenants to the New Barrage are grouped on the backfill of the diversion canal and are indicated on Album No. 9 for the service road variant and on Album No. 40 for the public road variant. They comprise separate administration buildings for the MOPWWR and EEA, and a workshop and a stores building for the hydropower plant. A tentative layout for both types of service buildings is given in Album No. 28.

## 4.11 HEAD REGULATORS

The Western and Eastern Head Regulators were constructed at the same time as the Barrage. The superstructures are of unreinforced concrete, and rubble masonry with a limestone facing, and are essentially the same type of construction as the Barrage. The Western regulator comprises six arched gates and the Eastern regulator three. The structures are built on portland cement concrete base slabs and the regulating gates are also similar in dimensions and operation to those on the Barrage.

The headpond-tailwater level differences under which the head regulators work are less severe than those at the Barrage and the downstream aprons are much shorter. Rehabilitation of the head regulators would therefore not involve remaining risks as envisaged with the Barrage structure and is therefore recommended instead of replacement by new structures.

In assessing the required rehabilitation measures for the core structure, a desk study to assess their stability was undertaken considering the future headpond level of 65.9 m asl associated with the New Barrage. The results for the Western Head Regulator are summarized below:

Piping: The exit gradient is unacceptable and remedial measures are required. This will comprise the construction of a filter layer immediately downstream of the existing concrete apron.

Uplift: The upstream apron and core structure are stable against uplift but the downstream apron is not. Remedial measures will comprise the construction of a surcharge concrete slab on the downstream apron.

Sliding: The core structure is unsafe against sliding, however, the addition of the surcharge slab described above will be sufficient.

The higher elevation of the floor slab of the Eastern Head Regulator results in the structure being safe against piping, uplift, and sliding.

Complete rehabilitation and refurbishment of the mechanical equipment of the head regulators is required. This includes gate slots, fixed-wheel gates, bottom sills in all vents, steel linings, sill beams, and sealing plates in all upstream and downstream stoplogs, and provision of one set of new stoplogs and two new gantry cranes. In addition, all lower fixed wheel gates of the irrigation regulators will be provided with electrically-driven hoists.

## 4.12 PREPARATORY WORKS

Construction of the New Barrage (or of the New Barrage without Hydropower) will require that infrastructure is adapted before its commencement. The main adaptations on the west bank will be a re-alignment of the Nag El Dawwa irrigation canal which supplies an area of about 1,200 feddan. Rehabilitation of the canal and of the small parallel road should reasonably be performed during the period of site installation and mobilization.

Simultaneous with the impoundment of the headpond between the existing Barrage and the New Barrage, the double leaf gates of the existing Barrage have to be removed. This is necessary to provide sufficient cross-sectional area so that the pond between the Barrages can be impounded under decreasing differential head. The remainder of the gate leafs can then be removed when this filling is complete.

The Naga Hammadi minihydro powerplant at the Turbin Canal will be closed before impoundment of the river reach between the existing and New Barrages. The number of units installed is 3 with a 1.9 MVA rated capacity of each generator. The minihydro powerplant is being refurbished as described in Appendix R with new electrical equipment and repair of mechanical parts and of some of the civil works.

The HPPEA has intentions to dismantle the plant and embedded steel parts and use them at the Zifta Barrage on the Damietta branch of the River Nile to establish a new generation station. Within the programme for modernization of the structures on the Damietta branch there would be an opportunity to erect the minihydro powerplant with new civil works within a side canal on the right abutment of the Zifta Barrage. Head conditions there would correspond to those applicable to the Naga Hammadi minihydro equipment.

If the intentions by the HPPEA prove to be technically and economically viable, then dismantling of the equipment, especially the hydraulic steel structures and embedded parts, should take place well in advance of headpond impoundment by the New Barrage.

## 4.13 CONSTRUCTION

Construction of the project is expected to start in mid-2000. Based on the schedule shown in Album No. 57, the estimated construction time is 5.5 years. This includes commissioning of the 4 units, so that they will be available for full commercial operation in January, 2006.

Details of the construction procedure are contained in Appendix T. Given the large total volume of 7.0 million $m^3$, the excavation works and related handling of material are key aspects in construction of the New Barrage. The principal idea is to handle the excavated material only once, wherever possible. This means that excavation is scheduled in such a way that the excavated material can be placed immediately at another location, without intermediate stock-piling. A typical example for this procedure is given in Figure 4-4. With the exception of the sandfill part of the closure dam and backfill of the upstream area of the diversion canal, this procedure was applied to every item of excavation and fill. Another important principle is that as much excavation material as possible is placed in the floodway fill area. As a result, the left bank of the project area is only disturbed where absolutely necessary, thus minimising the impact on agricultural land and its production.

For handling of excavation and fill, material flow charts have been included in Appendix T. The flow chart for earthworks is shown on Figure 4-3 while the sequence of material handling is documented on Figures T-1 to T-11 in Appendix T. The sequence of construction can be summarized in the following steps:

- Close the El-Dom floodway by filling in initial excavation material from the right bank to obtain safe, continuous access to the construction area.

- Excavate the diversion canal to groundwater level and use material for side dykes; stockpile on the west bank.

- Dredge diversion canal material to floodway and place riprap in the diversion canal.

- Prepare the right and left banks of the river in the area of the construction pit, including the seepage cutoff wall in these areas.

- Close river by upstream and downstream cofferdams and construct the seepage cutoff wall.

- Excavate material from the construction pit above water level and deposit in the floodway area as shown on Figure 4-4.

- Excavate the construction pit to foundation levels with simultaneous dewatering.

- Construct the permanent cutoff walls and concrete structures.

- Install steel structural works and generating units simultaneously with secondary concrete.

- Remove the cofferdams; open the navigation lock for river transport and the sluiceway for river discharge.

- Construct closure dam while passing river flow at low level through the sluiceway.

- Impound the headpond; test and commission generating units and steel structural components.

- Remove the gates from the Barrage.

An important constraint on construction planning is the requirement that navigation on the River Nile be maintained throughout construction of the New Barrage. During the first phase of construction (construction months 1 - 18: mobilization and construction of the diversion canal), river traffic can pass the site in the present river channel. A pipeline for transporting slurry from dredging operations will cross the River Nile at sufficient depth to not impair navigation. During the second phase (months 15 to 29 : cofferdams and construction pit), and until month 55 of the third phase of construction (month 29 to 63 m: main barrage works) river traffic will pass through the diversion canal in which flow velocities will be sufficiently low under all conditions of discharge. During the fourth phase of construction (month 51 to 58: closure of diversion canal) river traffic will pass through the new navigation lock from month 55 after removal of the cofferdams. Prior to the headpond being impounded, the entire river discharge will pass through the sluiceway at levels given by the rating curve. The navigation lock will be operated at a low level, thus requiring the base of the upstream approach apron to be set at a low level. The diversion canal will then be closed by construction of the closure dam. During the fifth phase of construction (month 58 to 63: tests and commissioning) the headpond will be impounded to the design level of 65.9 m asl and the navigation lock will take up its normal function.

By establishing a detailed sequence of the works and by minimizing the area of the construction pit and diversion canal, a considerable reduction in quantities and construction time has been achieved compared with the earlier estimates from the Interim Study. An essential item of cost saving was the reduction of the number of sluiceway gate openings from 8 to 7, while still being able to safely evacuate the emergency release.

It should also be mentioned that the construction conditions at the New Barrage site are less favorable compared with those encountered at the New Esna Barrage, where the construction pit could be established on an existing island and no diversion canal was required.

The site installation of the contractor, including batching plant, screening plant, small storage areas and workshops, will be temporary in nature, so that an area of El Dom Island near the construction site can be used. All facilities will have to be removed after the end of the construction period, when the relevant area is returned to the owners.

For the construction camp, and the further site establishment with offices workshops, equipment parks and storage areas, locations are proposed in desert land east of the Eastern Naga Hammadi Canal. Both areas are owned by the Government and no acquisition of private land will be required for their utilization. The balance of areas with perennial cropping lost due to construction of the project and gained in the floodway and the backfill of the diversion canal shows a net gain of 114.2 feddan.

## 4.14 CONSTRUCTION COSTS

The estimate of the capital investment cost of the New Barrage was prepared in accordance with normal practice for feasibility studies carried out for international lending institutes. Details of the procedure and results are presented in Appendix T. All costs are estimated for September 1995 price level.

Costs of the civil engineering works were estimated by measuring the principal work quantities based on the design drawings and on the basis of adequate construction procedures and scheduling. Four basic cost elements were developed, these being the direct cost of construction, eg the cost of permanent materials, the cost of depreciation of construction equipment, the operating cost of the equipment, and the cost of manpower.

Basic unit costs are then derived by dividing the direct cost obtained for the principal items of production, such as concrete production and placement, excavation and transport of earth materials, etc, by the quantities involved in the design. These basic unit costs are then further combined to develop the compound rates used in the cost estimate, eg compound rates for concrete include production, placement, formwork and curing. Finally, the contractor's own costs are added to obtain the unit costs as used in the cost estimate.

The unit costs are split into local and foreign currency cost portions. The local portion represents manpower, operation cost of equipment, materials available in Egypt and part of the overhead cost. Fuel is considered as local cost. Foreign costs are mainly the depreciation cost of construction equipment, imported material and majority of the overhead cost. The costs of manufacturing, transportation and installation of electrical and mechanical equipment in the powerhouse have been determined based on the Consultant's records of prices for similar equipment recently manufactured or tendered, supported by inquiries to manufacturers.

The construction costs of the New Barrage are then distributed over the construction period according to the schedule of works, as shown on Figure 4-5 and in Album No. 57, to obtain the cashflow of investment costs for the New Barrage presented in Table 4.1. The physical contingencies added to the costs were detailed according to an assessment of the quantity estimate, type of works and risks involved in the different components contributing to the New Barrage. The associated assumptions are given in Appendix U. Engineering costs for a FIDIC-based engineering contracts have been added to the total construction costs assumpting a percentage of 9.5 which is a usual order of magnitude for such projects. The estimate of the Total Barrage Cost with service bridge at cost level September 1995 amounts to 322.86 million US$, from which the portion of 216.04 million US$ is cost incurred in foreign currency.

In addition to the construction cost of the project with service bridge, the costs of project variants with low level and elevated public traffic bridges were estimated, as given in Appendix T, Table T13. From these figures, the additional cost for the project variants were calculated as given in Table 4.2.

## Table 4.1: NEW BARRAGE – CASH FLOW

Fuel = Local

| No | Item of Works or Cost | Amount Million US$ | | | Duration | Period | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | 2000 | | 2001 | | 2002 | | 2003 | | 2004 | | 2005 | |
| | | Local | Foreign | Total | Quarter | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign |
| **A** | **Civil works** | | | | | | | | | | | | | | | | |
| 1 | Site Installation | 8.49 | 2.11 | 10.60 | 2 | 4.67 | 1.16 | 0.85 | 0.21 | 0.85 | 0.21 | 0.85 | 0.21 | 0.85 | 0.21 | 0.42 | 0.11 |
| 2 | Existing Barrage Works | 1.05 | 0.39 | 1.44 | 2 | | | 0.53 | 0.19 | 0.53 | 0.19 | | | | | | |
| 3 | Diversion Canal | 4.18 | 11.97 | 16.16 | 5 | 0.84 | 2.39 | 2.51 | 7.18 | | | | | | | 0.84 | 2.39 |
| 4.1 | Construction Pit – Construction | 10.84 | 18.20 | 29.04 | 9 | | | 4.82 | 8.09 | 4.82 | 8.09 | | | | | 1.20 | 2.02 |
| 4.2 | Construction Pit – Dewatering | 0.40 | 1.62 | 2.03 | 8 | | | | | 0.05 | 0.20 | 0.20 | 0.81 | 0.15 | 0.61 | | |
| 5 | River Channel – Exc. & Protection | 5.74 | 18.87 | 24.60 | 12 | 0.48 | 1.57 | | | 0.96 | 3.14 | 1.43 | 4.72 | 1.31 | 6.29 | 0.96 | 3.14 |
| 6 | Right Bank Closure Dyke | 0.73 | 1.21 | 1.95 | 2 | 0.37 | 0.61 | | | 0.37 | 0.61 | | | | | | |
| 7 | Left Bank Protection Dyke | 0.86 | 1.04 | 1.90 | 1 | | | 0.86 | 1.04 | | | | | | | | |
| 8 | Closure Dam | 2.30 | 3.95 | 6.25 | 2 | | | | | | | | | | | 2.30 | 3.95 |
| 9 | Navigation Lock incl. Guide Walls | 8.40 | 7.52 | 15.92 | 6 | | | | | | | 5.60 | 5.01 | 2.80 | 2.51 | | |
| 10 | Sluiceway incl. End Abutment | 6.34 | 5.58 | 11.92 | 6 | | | | | | | 4.23 | 3.72 | 2.11 | 1.86 | | |
| 11 | Powerhouse incl. Piers | 12.25 | 10.77 | 23.01 | 6 | | | | | | | 8.16 | 7.18 | 4.08 | 3.59 | | |
| 12 | Service Buildings & Switchyard | 0.68 | 0.83 | 1.50 | 2 | | | | | | | | | | | 0.68 | 0.83 |
| | **TOTAL A** | **62.26** | **84.06** | **146.32** | | | | | | | | | | | | | |
| **B** | **Hydro-mechanical equipment** | | | | | | | | | | | | | | | | |
| 1 | Sluiceway | 3.71 | 9.64 | 13.36 | 5 | | | 0.37 | 0.96 | 0.74 | 1.93 | 1.34 | 3.47 | 0.8[illegible] | 2.31 | 0.37 | 0.96 |
| 2 | Navigation Lock | 1.49 | 3.04 | 4.53 | 1 | | | | | 0.15 | 0.30 | 0.30 | 0.61 | 1.04 | 2.13 | | |
| 3 | Irrigation Head Regulators | 1.06 | 0.71 | 1.77 | 4 | | | | | | | | | 1.06 | 0.71 | | |
| 4 | Powerhouse | 2.25 | 6.87 | 9.13 | 3 | | | | 0.69 | | 1.37 | | 2.29 | 2.25 | 2.52 | | |
| | **TOTAL B** | **8.52** | **20.26** | **28.79** | | | | | | | | | | | | | |
| **C** | **Mechanical equipment** | | | | | | | | | | | | | | | | |
| 1 | Bulb Turbines | 3.93 | 27.52 | 31.45 | 6 | | | | | | 2.75 | | 14.68 | 3.93 | 7.34 | | 2.75 |
| 2 | Mechanical Auxiliary Systems | 0.64 | 4.48 | 5.12 | 4 | | | | | | 0.45 | | 2.24 | 0.64 | 1.34 | | 0.45 |
| | **TOTAL C** | **4.57** | **32.00** | **36.57** | | | | | | | | | | | | | |
| **D** | **Electrical equipment** | | | | | | | | | | | | | | | | |
| 1 | Generators incl. Excitation | 3.20 | 18.11 | 21.30 | 4 | | | | | | 1.81 | | 3.62 | 3.20 | 10.86 | | 1.81 |
| 2 | Electrical Equipment | 4.66 | 8.65 | 13.30 | 6 | | | | | | 0.86 | | 4.03 | 3.72 | 2.31 | 0.93 | 1.44 |
| 3 | Transformers and Switchgears | 3.34 | 8.16 | 11.50 | 4 | | | | | | 0.82 | | 1.63 | 3.34 | 4.90 | | 0.82 |
| 4 | 220 kV Transmission line | 3.11 | 0.78 | 3.89 | 4 | | | | | | 0.08 | | 0.16 | 3.11 | 0.47 | | 0.08 |
| | **TOTAL D** | **14.29** | **35.70** | **49.99** | | | | | | | | | | | | | |
| **I** | **Basic Construction Cost** | **89.64** | **172.02** | **261.66** | | **6.35** | **5.73** | **9.93** | **18.37** | **8.46** | **22.82** | **22.11** | **54.38** | **35.09** | **49.95** | **7.70** | **20.76** |
| **II** | **Contingencies** | 11.57 | 21.61 | 33.18 | | 0.95 | 0.86 | 1.74 | 3.09 | 1.50 | 3.35 | 2.37 | 5.85 | 3.78 | 5.51 | 1.23 | 2.96 |
| **III** | **TOTAL CONSTRUCTION COST** | 101.21 | 193.63 | 294.84 | | 7.30 | 6.60 | 11.67 | 21.46 | 9.96 | 26.17 | 24.48 | 60.23 | 38.87 | 55.47 | 8.93 | 23.71 |
| **IV** | **Engineering** | 5.60 | 22.41 | 28.01 | | 0.40 | 0.76 | 0.65 | 2.48 | 0.55 | 3.03 | 1.35 | 6.97 | 2.15 | 6.42 | 0.49 | 2.74 |
| **V** | **TOTAL PROJECT COST** | **106.81** | **216.04** | **322.86** | | **7.71** | **7.36** | **12.32** | **23.94** | **10.51** | **29.20** | **25.83** | **67.20** | **41.02** | **61.88** | **9.42** | **26.46** |

HP COST XLS / Disbursement / NAT / 07 08 1997

Table 4.2: ADDITIONAL COST OF PUBLIC ROAD VARIANTS

| Bridge-Variant | Add. Cost of Bridge $10^6$ US$ | Add. Cost of Road Connections[1] $10^6$ US$ | Total Add. Cost $10^6$ US$ |
|---|---|---|---|
| Low Level Bridge | 0.87 | 2.81 | 3.68 |
| Elevated Bridge | 4.07 | 2.81 | 6.88 |

Note [1] : Highway Standard

The main dimensions and technical data of the New Barrage structures are summarized in Tables 4.3 to 4.8

Table 4.3: NEW BARRAGE - MAIN DIMENSIONS OF SLUICEWAY

| | | |
|---|---|---|
| Crest Level of Piers | m asl | 69.0 |
| Crest Level of Sill | m asl | 52.8 |
| Apron Level | m asl | 48.8 |
| Lowest Foundation Level of Apron | m asl | 45.3 |
| Total Length, including End Piers | m | 175.5 |
| Total Crest Length | m | 119.0 |
| Number of Piers/End Piers | | 6 / 2 |
| Width of Gate Openings/Piers | m | 17.0 / 4.0 |
| Number of Gates | | 7 |
| Type of Gates: segmental gates, hydraulically activated, with incorporated flaps | | |
| Crest Level of Gates when closed | m asl | 66.3 |
| Gate Height/Radius | m | 13.5 / 11.0 |
| Flap Gate Length/Height | m | 15.2 / 3.1 |
| Total Flap Crest Length | m | 106.4 |
| Max. Discharge through Sluiceway at 65.9 m asl | $m^3/s$ | 5,700 |
| Max. Discharge through Flaps at 66.8 m asl | $m^3/s$ | 710 |
| Stoplogs, Number of Elements per Gate | | |
| - Upstream | | 6 |
| - Downstream | | 5 |
| Stoplogs, Height of Elements | m | 2.25 |

Table 4.4: NEW BARRAGE - MAIN TECHNICAL DATA OF POWERPLANT

| | | |
|---|---|---|
| Generating Units: Number/Type | | 4/Kaplan Bulb |
| Rotational Speed | rpm | 73.2 |
| Generating Capacity: Total/per Unit | MW | 64.0 / 16 |
| Discharge Capacity: Total/per Unit | | |
| - Design | $m^3$ | 1,280/320 |
| - Maximum | $m^3$ | 1,842/460.5 |
| Diameter of Bulb/Runner/Draft Tube | m | 7.40 / 6.75 /10.8 |
| Intake Gross Area at Trash Rack | $m^2$ | 285 |
| Intake Gross Velocity | m/s | 1.12 |
| Turbine Canal: Diameter | m | 14.2 |
| Turbine Canal Area at Bulb | $m^2$ | 201 |
| Setting of Runner/Draft Tube Axis | m | 51.4 |
| Length of Draft Tube (Runner Axis to Tailwater Exit) | m | 33.8 |
| Outlet Area: Height/Width | m | 10.8/14.2 |
| Setting of Draft Tube Crest | m asl | 56.8 |
| Extreme Operable Tailwater Levels (min/max) | m asl | 57.1/63.2 |
| Extreme Operable River Discharges (min/max) | $m^3/s$ | 3,520 |
| Powerhouse: Total Length between Piers | m | 70.8 |
| Width of Twin Unit Block | m | 31.4 |
| Total Length of Unit Block | m | 72.0 |
| Max. Overall Height of Unit Block | m | 31.4 |
| Crest Level of Powerhouse | m asl | 69.0 |
| Crest Level of Road Platform | m asl | 69.0 |
| Powerhouse Hall Floor Level | m asl | 61.2 |
| Intake Floor Level | m asl | 44.3 |
| Draft Tube Floor Level | m asl | 46.0 |
| Lowest Foundation Level | m asl | 38.6 |
| Powerhouse Hall Cranes: Number | | 2 |
| Lifting Capacity of Hoists | t | 10 / 2.5 |
| Net Head at River Discharge 1842 $m^3/s$ (no flow over sluiceway) | m | 4.32 |
| Net Head at River Discharge 350 $m^3/s$ | m | 7.97 |
| Min. Net Head at River Flood Discharge 3.520 $m^3/s$ (flow through powerhouse & sluiceway) | m | 2.4 |
| Generator Capacity | MVA | 18.8 |
| Generator Reactive Power | MW | 16 |
| Power Factor | | 0.85 |
| Generator Voltage | kV | 11 |
| Main Transformers: No./Type | | 3 / 3 Phase |
| Voltage | kV | 11 / 132 |
| Switchgear: Type | | Conventional |
| Bays - from Main Transformer | | 2 |
| - to Overhead Line | | 2 |
| - Bus Coupling | | 1 |
| Overhead Line: Type | | Double Circuit |
| Voltage | kV | 132 |
| Emergency Gates Upstream: Type | | Fixed Wheel |
| Clear Width | m | 14.2 |
| Height of Element | m | 7.7 |
| No. of Elements/Gate | | 2 |
| Emergency Gates Downstream: Type | | Stoplogs |
| No. of Elements | | 2 |

Table 4.5: NEW BARRAGE - MAIN DIMENSIONS OF NAVIGATION LOCK

| | | |
|---|---|---|
| Lock Chamber: Usable Length/Width | m | 170.0 / 17.0 |
| Levels of US/DS Sills | m asl | 56.6 / 54.90 |
| Bottom Level of Chamber | m asl | 54.90 |
| Navigable Tailwater Levels: max. 2900 $m^3/s$ | m asl | 62.5 |
| min. 350 $m^3/s$ | m asl | 57.90 |
| Clearance at Min. Navigable River Flow | m | 3.0 |
| Crest Level of Structure | m asl | 69.0 |
| Foundation Level: Normal/Maximum | m asl | 50.5 / 46.5 |
| Total Structural Length | m | 217.5 |
| Width at Foundation Level | m | 32 |
| Upstream Gate: Type | | Double Leaf/ Vertical Lift |
| Number of Leaves | | 2 |
| Total Height | m | 9.8 |
| Crest Height for Lock Operation | m asl | 62.7 |
| Time for Opening/Closing | min | 4 to 5 |
| Downstream Gate: Type | | Mitre |
| Height | m | 11.5 |
| Time for Opening/Closing | min | 4 to 5 |
| Clearance of Service Bridge above Max. Navigable Q | m | 5.0 |
| Crest of Road Bridge (approx.) | m asl | 69.0 |
| Length /Area of Filling Duct | $m/m^2$ | 30.0 x 9.0 |
| Length/Area of Emptying Duct | $m/m^2$ | 82.0 x 9.0 |
| Approx. Dimensions of Fixed Wheel Gates | m | 3.2 x 3.2 |
| No. of Gates | | 4 |
| Max. Filling/Emptying Time | min | 10 / 11 |

Table 4.6: PIER AND GUIDE WALL STRUCTURES

| | | |
|---|---|---|
| Powerhouse Abutment Pier: | | |
| Contains: Entrance to Powerhouse | | |
| Powerhouse Hall | | |
| Staircases and Elevators | | |
| Access Shaft to Turbine Shaft Gallery | | |
| Cable Shaft | | |
| Workshop and Storerooms | | |
| Relay and Protection Room | | |
| Access Shaft to Turbines and Generators | | |
| Length/Max Width | m | 155.0 / 17.5 |
| Foundation Level | m asl | 38.5 - 46.0 |
| | | |
| Intermediate Pier: | | |
| Contains: Entrance to Powerhouse | | |
| Powerhouse Hall | | |
| Diesel Plant | | |
| Hatches to Drainage Sump | | |
| Access Shaft to Drainage Sump | | |
| Drainage, Dewatering, and Oil Separating Sumps | | |
| Access Shaft to Turbines and Generators | | |
| Staircases and Elevators | m | 138.4 / 17.5 |
| Length/Max. Width | m asl | 43.5 - 48.8 |
| Foundation Level | | |
| | | |
| Sluiceway Abutment Pier: | | |
| Contains: Emptying Canal and Outlet | | |
| Length/Max Width | m | 123.0 / 15.0 |
| Foundation Level | m asl | 45.2 - 43.2 |
| | | |
| Connecting Structure: | | |
| Contains: Lock Filling and Emptying | | |
| Canals & Gates, Filling Inlet | | |
| Length/Width | m | 68.0 / 17.5 |
| Foundation Level | m asl | 45.2 |
| | | |
| Navigation Lock U/S Guide Wall: | | |
| Approx. Length | m | 120.0 |
| Approx. Height above Foundation | m | 18.5 |
| | | |
| Navigation Lock D/S Guide Wall: | | |
| Approx. Length | m | 120.0 |
| Approx. Height above Foundation | m | 15.0 |

Table 4.7: <u>EMBANKMENT STRUCTURES</u>

| | | |
|---|---|---|
| 1. Closure Dam | | |
| Type: Sandfill between Rockfill with Central Diaphragm Wall | | |
| Foundation Level/Crest Level | m asl | 52.0 / 69.0 |
| Crest Width | m | 15.0 |
| Max. Height above Foundation | m | 17.0 |
| Crest Level of Rockfill Embankments | m asl | 62.0 |
| Slopes of Rockfill Embankments | | 1:2 |
| Slopes of Sandfill Embankments | | 1:3 |
| Approx. Max. Width of Foundation | m | 113 |
| Approx. Length at Crest Level | m | 220 |
| | | |
| 2. Right Bank Closure Dyke | | |
| Type: Sandfill | | |
| Approx. Height above Foundation | m | 1.5 to 8.0 |
| Crest Level | m asl | 69.0 |
| Approx. Length of Embankment | m | 2,600 |
| Slopes, River Side/Shore Side | | 1:3 / N.A. |
| | | |
| 3. Left Bank Protection Dyke | | |
| Type: Sandfill | | |
| Approx. Height above Foundation | m | 2.0 |
| Crest Level | m asl | 69.0 |
| Approx. Length of Embankment | m | 2,900 |
| Slopes, River Side/Shore Side | | 1:3 / 1:3 |
| | | |
| 4. Temporary Cofferdams at Construction Pit | | |
| Type: Sandfill with Diaphragm Wall and d/s Rockfill | | |
| Lowest Foundation Level/Crest Level | m asl | 51.0 / 65.0 |
| Max. Height above Foundation | m | 14.0 |
| Crest Width | m | 18.0 |
| Crest Level of Rockfill Embankment | m asl | 62.0 |
| Slopes of Rockfill Embankment | | 1:2 |
| Slopes of Sandfill Embankment | | 1:3 |
| Approx. Max. Width of Foundation | m | 99 |
| Approx. Crest Lengths, u/s and d/s | m | 318 / 307 |

Table 4.8: STRUCTURES SPANNING THE ENTIRE BARRAGE

| | | |
|---|---|---|
| 1. Temporary Cutoff Walls around Construction Pit | | |
| Max./Min./ Level | m asl | 67.5 / 65.0 |
| Approx. Max. Depth | m | 58.0 |
| Approx. Total Length | m | 310 |
| Thickness | m | 1.0 |
| | | |
| 2. Permanent Cutoff Walls | | |
| Max. Level at Closure Dam & Banks | m asl | 67.5 |
| Max. Level at Navigation Lock | m asl | 46.5 |
| Max. Level at Powerhouse | m asl | 39.6 |
| Max. Level at Sluiceway | m asl | 47.2 |
| Approx. Min. Level | m asl | 33 |
| Approx. Total Length | m | 710 |
| Thickness | m | 0.8 |
| | | |
| 3. Main Gantry Cranes | | |
| Length of Crane Runway | m | 310 |
| Span between Rails | m | 21.0 |
| No. of Cranes | | 2 |
| Lifting Capacity of Hoists | t | 63 / 20 |
| Max. Lift Level | m asl | 79.5 |
| | | |
| 4. Roads and Bridges | | |
| 4.1 Service Road | | |
| Approx. Length /Width of Road on Main Barrage Structures | m | 310/4 |
| Length of Bridge over Sluiceway | m | 143/4 |
| Length/Width of Bascule Bridge over Navigation Lock | m | 17/7 |
| 4.2 Low Level Public Road | | |
| Approx. Length /Width of Road on Main Barrage Structures | m | 310/9 |
| Length of Bridge over Sluiceway | m | 143/9 |
| Length/Width of Bascule Bridge over Navigation Lock | m | 17/9 |
| 4.3 Elevated Public Road | | |
| Approx. Length /Width of Road on Main Barrage Structures | m | 310/9 |
| Length of Concrete Ramp on Closure Dam | m | 100 |
| Length/width of Concrete Ramp at Navigation Lock | m | 60 |
| Length of Earthfill Ramps on Right Bank/Left Bank | m | 230/189 |

# 5. NEW BARRAGE WITHOUT HYDROPOWER COMPONENT

## 5.1 PURPOSE OF EVALUATION

The New Barrage outlined in Chapter 4 serves two main purposes, namely to guarantee the continued supply of irrigation and generation of hydropower. From the economic viewpoint, once the decision is made for implementation of the New Barrage, investment costs which have to be borne by the hydropower component can be separated from the overall cost by deducting the cost of a New Barrage without Hydropower. Although it is not intended to constructed the latter, this barrage would serve the purpose of irrigation and river navigation, and would be located at the same site as the New Barrage. Referred to as the Base Case in the Economic and Financial Analyses, Chapters 9 and 10, the New Barrage without Hydropower would require a headpond level of 65.9 m asl, commensurate with the design headpond level of the existing and New Barrages.

## 5.2 LAYOUT

The layout of the New Barrage without Hydropower, shown on Album Nos. 52 and 53, is generally comparable to the New Barrage layout described in Chapter 4 with some modifications to the locations of the project components. Firstly, the sluiceway, being the long-term water discharging structure, is relocated to the centre of the river channel. This allows the navigation lock to be moved some 100 m closer to the river channel, significantly reducing the excavation of its upstream and downstream approaches when compared with the New Barrage layout.

The relative location and orientation of the navigation lock and sluiceway with the connecting structure are the same as for the New Barrage. However, the left abutment pier of the sluiceway is now in the position of the abutment pier of the powerplant in the "with Hydropower" layout (compare Album Nos. 9 and 53). This allowed the diversion canal, which had been optimized in size and shape for the New Barrage layout, to be maintained.

The layout was developed with the service bridge only, as only this case was considered in the economic and financial evaluations of hydropower.

The design criteria for the New Barrage without Hydropower are principally the same as given for the New Barrage in Chapter 3. Furthermore, the backwater effect upstream of the New Barrage without Hydropower remains unchanged compared with the effects from the project with Hydropower. The same applies for the headpond level rise during the emergency flood.

Some specific aspects of the project layout without hydropower are discussed below:

*Sluiceway*

At the New Barrage, the river flow normally passes through the hydropower plant, where energy resulting from the hydraulic head is converted into electrical energy. At the New Barrage without Hydropower, the energy of the river flow is not used and must be permanently dissipated when the flow passes the sluiceway under submergence by the tailwater. Hence, there will be a high amount of energy which must be permanently dissipated across the relatively small width of the downstream apron and adjacent river. It should be noted that at the existing Barrage this energy is dissipated on and downstream of an 820 m wide apron; however, with almost no submergence by the tailwater. In contrast, the width of the downstream apron proposed for the New Barrage without Hydropower is only approximately 170 m, but submergence by the tailwater is considerable.

In assessing the appropriate width and specific discharge of the permanently operating sluiceway, it could not be concluded whether the width of the sill should be wider than for the "with Hydropower" case and if protection of the downstream exit channel and riverbed is adequate. However, considering maintenance would normally be carried out during the dry season and the emergency release from HAD would not be discharged without prior notice, the sluiceway was designed with

the same number of gate openings as in the "with Hydropower" case. This resulted in the adoption of 7 gate openings each of 17m width.

It should be noted that in contrast to the New Barrage, it is the foundation level of the sluiceway sill which determines the lowest level of the construction pit and in turn areal extent of the embankment crest as shown on Album No. 54.

All dimensions of the hydromechanical and steel structural equipment for the sluiceway of the New Barrage without Hydropower are essentially unchanged from those of the New Barrage as outlined in Chapter 4. Although for regulation of river flows the sluiceway gates do not need to be equipped with flaps, for the purpose of weed control and discharge of debris, they are foreseen on 3 of the 7 gates.

For maintenance of the gates, setting of stoplogs, and also use during construction, one gantry crane of similar capacity although of less extension to that proposed for the New Barrage is foreseen. The crane runway beams are positioned as for the New Barrage and the service bridge is at the same distance downstream.

*Construction Pit for the Main Concrete Works*

The construction pit is shown on Album No. 54. Its length is determined by the length of the navigation lock and associated upstream and downstream guidewalls, and is hence similar to that of the New Barrage. The left abutment pier of the sluiceway for the "without Hydropower" case replaces the abutment pier of the powerplant in the New Barrage layout. This results in an identical location and shape of the construction pit contours and the diversion channel for the two layouts. At the right abutment, the location of the navigation lock is now some 100 m closer to the river channel, which results in a considerable reduction in the width of the pit compared with the New Barrage layout (compare Album Nos. 11 and 54).

The depth of the construction pit is, however, determined by the lowest foundation depth of the project components. This is associated with the sluiceway apron at an elevation of 45.20 m asl. This is some 6.5 m above the bottom level required for the foundation of the hydropower plant, with an approximate 35m reduction in the width of the side slopes of both banks of the construction pit.

The total excavation for the construction pit and length of the surrounding seepage cutoff wall reduce to some 39% and 85% respectively of those for the New Barrage.

*Embankment and Excavation Works*

Other main works involved in the layout are embankments and excavation of the diversion canal and approaches from and to the navigation lock and sluiceway.

There is essentially no change in the permanent embankment structures for this layout compared with the New Barrage. In view of the much smaller excavation works totaling only about 5.2 million $m^3$ compared with 7.0 million $m^3$ for the New Barrage, the area of reclaimed land on the former floodway to the right of El Dom Island (shown on Album No. 52) will be 146.9 feddan only instead of 229.6 feddan for the New Barrage. This reduction is partly compensated by the larger unaffected areas of El Dom Island resulting from the reduced areal requirement for upstream and downstream approaches to the navigation lock (see Album Nos. 52 and 53).

As the permanent left bank material deposit stems from the excavation of the diversion canal, and the quantities of the canal remain unchanged compared with the New Barrage layout, the amount of deposited earthfill material and backfill into the upper portion of the diversion canal remains the same as for the New Barrage.

deposited earthfill material and backfill into the upper portion of the diversion canal remains the same as for the New Barrage.

The length of permanent cutoff walls required in the layout is only reduced by some 86 m, which is some 20% of the requirement for the New Barrage layout.

## 5.3 CONSTRUCTION PROCEDURE, QUANTITIES AND COSTS

The construction schedule shown in Album No. 58 for the construction and diversion procedure is essentially the same as described for the New Barrage (Album No. 57). The reduction in construction time when compared to the New Barrage is only 0.5 years, reflecting the smaller excavation of the construction pit, a reduced period for construction of the concrete works, and elimination of the commissioning time for the generating units. The total construction time of the New Barrage without Hydropower is 57 months.

The schedule of construction quantities is presented in Appendix T, together with a material flow chart for the excavation and embankment works. The unit costs of civil works construction adopted for the New Barrage have been applied without change to the "without Hydropower" layout. The percentage estimates for the physical contingency for different items of work also remains the same as for the New Barrage and is outlined in Appendix U. The cost of construction camp and site installation was reduced from the New Barrage in relation to the total cost of the works and the reduction in construction time.

The construction costs of the New Barrage without Hydropower are then distributed over the construction period according to the schedule of works to obtain the cashflow of investment costs. This is presented in Table 5.1. In this cashflow, fuel cost is considered to be in local currency, and a percentage is added for engineering and administration.

The estimate of the total construction costs for the New Barrage without Hydropower amounts to 170.15 million US$ at level September, 1995. The foreign currency component is estimated to be 106.38 million US$.

## Table 5.1: BARRAGE WITHOUT HYDROPOWER – CASH FLOW

Fuel = Local

| No | Item of Works or Cost | Amount Million US$ | | | Duration | Period | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | 2000 | | 2001 | | 2002 | | 2003 | | 2004 | | 2005 | |
| | | Local | Foreign | Total | Quarter | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign |
| **A** | **Civil works** | | | | | | | | | | | | | | | | |
| 1 | Site Installation | 6.56 | 1.64 | 8.20 | 2 | 3.61 | 0.90 | 0.66 | 0.16 | 0.66 | 0.16 | 0.66 | 0.16 | 0.66 | 0.16 | 0.33 | 0.08 |
| 2 | Existing Barrage Works | 1.05 | 0.39 | 1.44 | 2 | | | 0.53 | 0.19 | 0.53 | 0.19 | | | | | | |
| 3 | Diversion Canal | 4.10 | 11.72 | 15.82 | 5 | 0.82 | 2.34 | 2.46 | 7.03 | | | | | | | 0.82 | 2.34 |
| 4.1 | Construction Pit -- Construction | 9.24 | 15.29 | 24.52 | 9 | | | 4.11 | 6.79 | 4.11 | 6.79 | | | 1.03 | 1.70 | | |
| 4.2 | Construction Pit – Dewatering | 0.33 | 1.53 | 1.86 | 6 | | | | | 0.06 | 0.25 | 0.22 | 1.02 | 0.06 | 0.25 | | |
| 5 | River Channel – Exc. & Protection | 4.27 | 14.51 | 18.77 | 8 | 0.53 | 1.81 | | | 1.07 | 3.63 | 1.60 | 5.44 | 1.07 | 3.63 | | |
| 6 | Right Bank Closure Dyke | 0.73 | 1.21 | 1.95 | 3 | 0.24 | 0.40 | | | 0.49 | 0.81 | | | | | | |
| 7 | Left Bank Protection Dyke | 0.86 | 1.04 | 1.90 | 1 | | | 0.86 | 1.04 | | | | | | | | |
| 8 | Closure Dam | 2.30 | 3.95 | 6.25 | 2 | | | | | | | | | 1.15 | 1.98 | 1.15 | 1.98 |
| 9 | Navigation Lock incl. Guide Walls | 8.40 | 7.52 | 15.92 | 5 | | | | | | | 6.72 | 6.01 | 1.68 | 1.50 | | |
| 10 | Sluiceway incl. End Abutment | 8.18 | 7.27 | 15.45 | 6 | | | | | 1.36 | 1.21 | 5.45 | 4.85 | 1.36 | 1.21 | | |
| 12 | Service Buildings | 0.35 | 0.40 | 0.75 | 2 | | | | | | | | | | | 0.35 | 0.40 |
| | **TOTAL A** | **46.37** | **66.47** | **112.84** | | | | | | | | | | | | | |
| **B** | **Hydro-mechanical equipment** | | | | | | | | | | | | | | | | |
| 1 | Sluiceway | 3.95 | 11.02 | 14.97 | 5 | | | 0.40 | 1.10 | 0.79 | 2.20 | 1.74 | 4.85 | 0.63 | 1.76 | 0.40 | 1.10 |
| 2 | Navigation Lock | 1.49 | 3.04 | 4.53 | 1 | | | | | 0.15 | 0.30 | 0.30 | 0.61 | 1.04 | 2.13 | | |
| 3 | Irrigation Head Regulators | 1.06 | 0.71 | 1.77 | 4 | | | | | | | | | 1.06 | 0.71 | | |
| | **TOTAL B** | **6.50** | **14.77** | **21.28** | | | | | | | | | | | | | |
| **D** | **Electrical equipment** | | | | | | | | | | | | | | | | |
| 1 | Connection to NHB Substation | 0.36 | 1.24 | 1.60 | 4 | | | | | | | | | 0.36 | 1.24 | | |
| **I** | **Basic Construction Cost** | **53.24** | **82.48** | **135.72** | | **5.21** | **5.46** | **9.00** | **16.33** | **9.20** | **15.56** | **16.69** | **22.94** | **10.09** | **16.28** | **3.04** | **5.91** |
| **II** | **Contingencies** | 7.58 | 12.09 | 19.67 | | 0.78 | 0.82 | 1.56 | 2.74 | 1.51 | 2.54 | 1.83 | 2.78 | 1.43 | 2.31 | 0.48 | 0.91 |
| **III** | **TOTAL CONSTRUCTION COST** | 60.82 | 94.57 | 155.39 | | 5.99 | 6.28 | 10.57 | 19.07 | 10.71 | 18.10 | 18.51 | 25.72 | 11.52 | 18.58 | 3.52 | 6.82 |
| **IV** | **Engineering** | 2.95 | 11.81 | 14.76 | | 0.29 | 0.78 | 0.51 | 2.38 | 0.52 | 2.26 | 0.90 | 3.21 | 0.56 | 2.32 | 0.17 | 0.85 |
| **V** | **TOTAL PROJECT COST** | **63.77** | **106.38** | **170.15** | | **6.28** | **7.07** | **11.08** | **21.46** | **11.23** | **20.36** | **19.41** | **28.93** | **12.08** | **20.90** | **3.69** | **7.67** |

NO COST XLS / Disbursement / NAT / 07 08 1997

# 6. CAPACITY AND ENERGY GENERATION AVAILABLE FROM THE NEW BARRAGE

## 6.1 OPERATION OF THE HYDROPOWER PLANT

The discharges in the River Nile are now totally controlled by the HAD and releases are predetermined annually by the MOPWWR in line with projected water demands downstream. Although the total volume of discharge has generally remained consistent from year to year since commissioning of the HAD in the late 1960s (apart from the drought period in the 1980s), the seasonal distribution of flows has shown some variation. Particularly since 1994 there has been an increase in summer discharges (June to August) and a reduction in winter flows (October to February) from those observed previously (discussed in Section 2.2.2 and in detail in Appendix H). The MOPWWR indicated this strategy will be indicative of their likely future release strategy and so was appropriate for the assessment of energy generation for the New Barrage. The annual average discharge was estimated to be 1,380 $m^3/s$.

In addition, the Ministry also indicated that for short periods during the year it may be necessary to draw the headpond down from the proposed level of 65.9 m asl to release additional flow to meet increased short-term demands further downstream. This would be likely to be limited to a total period of some 30 days annually. The timing of such releases is, however, uncertain and it was therefore agreed with the MOPWWR that for the purposes of energy generation, periodic drawdowns be introduced for three 10-day periods. The magnitude of the adopted drawdown was 0.25 m during periods at the beginning of March and May and in mid-October.

Benefits attributable to a hydropower scheme are determined on the basis of two components, namely, average annual energy generation and the dependable capacity. The latter is defined as the power which can reliably be supplied to satisfy system demand for a nominated percentage of time. From the viewpoint of the existing system, this was adopted as 90%. The firm energy is then the annual energy generated with this dependable capacity, taking into account of course the reduction in generated power during the remaining 10% of the year when the dependable capacity cannot be provided. In quantifying these components for the proposed hydropower plant at Naga Hammadi only run-of-river operation was considered.

The run-of-river mode would involve operating the powerplant at a constant headpond level throughout the year (apart from the limited periods when the headpond level would be drawn down as discussed above). The rate of turbined flow would be equivalent to the observed discharge of the River Nile except in the maximum irrigation months (June, July and August) when it is limited by the capacity of the plant. In this operating mode, the installed capacity of the powerplant is fully utilised only in the range of rated flow and head which, based on the discussion below and as shown in Figure 2-2, occurs only over a short period of the year.

With a constant headpond level, the hydropower plant will not be able to adapt generation to the daily variation of the demand. Hence it will only be used for two purposes:

- to supply base load to the system at its firm capacity, and
- to add non-firm energy generation to the system and save fuel from thermal generation which would otherwise have been required.

The generation by the Naga Hammadi hydropowerplant will be fully governed by the flow conditions associated with the release pattern from the HAD. As a result energy generation will reduce below maximum levels during two periods:

- October to February during the low discharge period, including winter closure when the irrigation demands reduce to their minimum for approximately 6 weeks in December and January, and

- June to August in the peak irrigation season when higher discharges reduce the generation head.

The daily load curves for the different seasons indicate demands through the day show some minor seasonal variation but the shape remains essentially unchanged with a five hour evening peak.

## 6.2 ENERGY AND CAPACITY FROM THE HYDROPOWER PLANT

Energy calculations were performed for the purpose of optimizing the installed capacity of the hydropower plant (see Appendix Q) and for economic evaluation (as described in Chapter 9).

They were based on an average ten-day series of observed flows recorded immediately downstream of NHB based on the recent two year period from September, 1994 to August, 1996 (see Appendix H). However, the operation of the HAD now incorporates a significant reduction in releases during the month of January during the so-called period of winter closure. Following discussions with the MOPWWR, the flow sequence was modified and a constant value of 350 $m^3$/s adopted for the mid-10 day period in January. As shown on Figure 2-2, this reduction in flows has only a minor impact on the flow duration curve at the very low end of the discharge range.

In determining the power and energy generation based on this flow sequence, the tailwater rating curve shown on Figure 2-4 together with a turbine efficiency chart for the proposed bulb-type turbine were utilised. In addition, the combined efficiencies of the generators and transformers, and transmission losses to Naga Hammadi Substation were also considered. Finally, the calculations also incorporated hydraulic head losses resulting from the flow through the civil structures of the powerplant which increased to a maximum of 0.45m at full turbine discharge capacity.

Bulb units were selected due to their high overall efficiency and good operational characteristics to generate at part load and extreme low heads which prevail during high river flows. Optimisation of the number of units and blades resulted in the adoption of four bulb turbines, each unit having three blades. The selection of the number of blades was based on both the marginally higher energy generation when compared to units with four blades and the higher rotational speed of the units, an advantage for sizing and optimisation of the generator. Four units was found to be economically more advantageous than six. Optimisation of the installed capacity was based on a marginal generation cost analysis which compared the incremental cost of additional generation (based on increasing turbine discharge and so utilising a larger proportion of the flow duration curve) with the cost of equivalent energy generated by thermal plant located in Cairo. This resulted in a total installed capacity of 64 MW and a rated discharge of 1,280 $m^3$/s. The turbines can discharge up to a maximum of 1,840 $m^3$/s. The time series of generated power is presented in Figure 6-1. Also included is the associated time series of discharges indicating the reduction in generated power during the periods of high flow (reduced head) and low flow during winter. The power duration curve is shown on Figure 6-2.

The generator and transformer efficiencies were adopted as 97% and 99.5% respectively while station use at the powerhouse was of the order of 0.5%. Finally, transmission losses between the power station and Naga Hammadi substation amounted to 1.1 GWh/y which represents some 0.24% of the annual generation. Incorporating these losses, the estimated energy and dependable capacity for the Naga Hammadi powerplant are:

| | | |
|---|---|---|
| Total annual energy | = | 462.6 GWh/y |
| Dependable capacity | = | 41.8 MW |
| Firm energy | = | 326.8 GWh/y |

The dependable capacity, the maximum power available from Naga Hammadi for 90% of the time, is defined from Figure 6-2 after allowing for generator and transformer efficiencies.

## 6.3 EFFECT OF DOWNSTREAM RIVERBED DEGRADATION ON ENERGY GENERATION AT NAGA HAMMADI

Long-term riverbed degradation downstream of the Naga Hammadi Barrage was estimated using the HEC-6 sediment transport model, as described in Chapter 2 and Appendix K. Based on the results of those studies, revised tailwater rating curves for the New Barrage site were derived at intervals of ten years. These were then used to derive energy estimates following future river degradation downstream for the headpond level of 65.9 m asl.

Overall, the increase in average annual energy generation from Naga Hammadi station resulting from the predicted degradation downstream of the New Barrage over the following 50 years is estimated to be around 15%. The average annual energy generation 50 years after implementation of the Project is estimated to be 506.3 GWh (prior to deducting transmission losses). In addition, the analyses have highlighted the relatively significant increase in energy which is likely to result between now and the earliest likely commissioning of the project. Adopting a commissioning date in the fiscal year 2005/2006, the additional energy resulting from the downstream degradation is around 20 GWh/y, or some 4% higher than the value utilized in the economic evaluations for the project.

In a sensitivity analysis, the present value of energy generation at the time of implementation was assessed. This involved the estimation of a 50 year sequences of generated energies (incorporating the effects of degradation) which were then discounted to year 0 and the present value of energy generation determined. From this calculation, the ratio of the present value of series with degradation to the series without degradation was calculated to be 1.06 at a discount rate of 6%.

## 6.4 POSSIBLE EFFECT OF UPSTREAM ABSTRACTIONS BY TOSHKA PUMPING STATION

A tender is currently underway for a new development being considered by the Egyptian Government which will involve pumping water directly from Lake Nasser just north of the Toshka emergency spillway to allow the development of the large scale 'New Valley' agricultural scheme. A preliminary attempt has been made to quantify the impact of this abstraction on future downstream releases from HAD in order to assess the effect on long-term energy generation at the New Naga Hammadi Barrage.

It is likely, however, that the 'New Valley' project will be expanded in line with agricultural development, and associated settlement and industry. Therefore, the maximum discharge (300 $m^3/s$) was assumed to occur only after 20 years. A peak discharge of 90 $m^3/s$ was assumed at commissioning, and 160 $m^3/s$ and 230 $m^3/s$ after five and ten years respectively. Annual time series of 10-day average abstractions were derived for these peak pumping discharges based on the observed seasonal discharge pattern of the Western Naga Hammadi canal immediately upstream of the existing Barrage (which has an annual peak discharge of around 140 $m^3/s$). These 10-day flow series were then subtracted directly from the adopted sequence at the New Barrage used in the energy calculations outlined in the Section 6.2 and annual estimates of energy generation derived for the New Barrage for each time interval (that is the year of commissioning of the Toshka pumping station, and at intervals of 5, 10, and 20 years).

Assuming the Toshka pumping station is commissioned in the same year as the New Barrage, the reductions in energy generation range from 0.5% at the start of operation to 3.3% after 20 years. This reflects the change in flows in the River Nile which, in comparison to those now observed at Naga Hammadi, would have reduced by 4.5% and 15% for these same periods.

The increased pumping during summer will actually result in higher power and energy generation at Naga Hammadi during this season than would occur with no development in the 'New Valley'. This energy gain results from the flows remaining high, but due to lower tailwater levels the net head increases. However, this energy gain is more than offset by the overall reduction in flows and energy generation during the remainder of the year as shown by the reducing annual totals.

## 6.5 REQUIREMENT FOR ADDITIONAL CAPACITY AND ENERGY AT THE TIME OF PROJECT IMPLEMENTATION

As documented in Appendix N, the power demand growth in EEA's Unified Power System (UPS) averages around 6%. There is a similar growth in the energy demand, maintaining the system load factor at around 73%.

EEA uses and updates a load forecast which was established on the basis of macroeconomic factors, such as growth of population and gross domestic product. The predicted growth rates for the electricity demand appear to be high but may be justified by economic growth through ongoing privatization in the industrial and banking sectors.

The growth of electricity demand is almost equally distributed over the 7 supply zones of the UPS shown on Figure 6-3, whereas the additional installation of thermal powerplant takes place only in the northern zones. In South Upper Egypt, the system of the HAD and the downstream Aswan I and II hydropower plants presently produce about 21% of the total energy in the UPS, which must then be transmitted to the main consumer zones located in Cairo and the Delta.

However, with the well-balanced overall demand growth in Egypt, there has been a significant increase of requirements in the South. With implementation of industrial proje[illegible] in South Upper Egypt and the planned implementation of the 350 MW Toshka pumping station for the large development in the 'New Valley' area, the consumption in the South Upper Egypt will significantly increase. This will result in a further reduction of energy flow to the northern zones; in effect, less excess energy is already now available for transfer from Upper Egypt than was previously the case. To compensate, the generation system in the North is being further developed through the installation of thermal powerplant of high capacity.

The future development of thermal powerplant will take place near the centres of exploration for natural gas or in areas which can be supplied by gas pipelines of reasonable length and where cooling water is available. The development of large gas-fired thermal plant will therefore take place in all districts of the UPS except North and South Upper Egypt.

From energy flow charts supplied by EEA, it is concluded that the region in which the Naga Hammadi Barrage is located consumes some 2.8 times the energy it is generating, resulting in the import of electricity over long distances from Aswan and Cairo. Generation of electricity in the Naga Hammadi region would therefore be an advantage for the whole system, given the reduction in transmission losses associated with the energy no longer required to be transferred to this region

To assess the need for capacity in the UPS at the earliest time when the New Barrage hydropower plant could be made available, EEA's present generation system was analyzed in terms of installed capacity and firm dependable output. This is outlined in Table N4, Appendix N. The firm dependable output is estimated to be about 10,800 MW in 1995. This signifies an overcapacity of some 25% which is substantially above normal reserve margins for maintenance requirements and

loss of load. However, EEA's ongoing system generation planning is based on the principle of reducing the present overcapacity and to come to normal system reserve margins. It is also based on the requirement to retire old thermal plant of small capacities and with low thermal efficiencies with an economic life that has already expired. With this requirement, EEA established a short-term system expansion schedule up to the fiscal year 2001-2002. The earliest commissioning of the New Barrage hydropower plant would be in the fiscal year 2005-2006. The short-term system expansion plan contains three large steam powerplant additions to the system, which are committed or already under construction, namely. Kureimat Steam (4 x 300 MW), Sidi Krir (2 x 300 MW) and Ain Moussa (2 x 300 MW). A further system addition would be an extension of the Sidi Krir plant by another 2x300 MW.

Further thermal-system additions have not been definitely scheduled to date, and EEA is expecting that the Naga Hammadi Hydropower Plant will be commissioned in the fiscal year 2005-2006. The development of further thermal plants in the UPS would fully consider the hydropower installation by which the installation of thermal plants could be limited.

Finally, a balance was produced between the projected system demand and the firm dependable capacity of all existing plant plus committed system additions and with the New Barrage minus thermal plant retirement, for the fiscal year 2005-2006. This balance clearly shows that EEA requires thermal plant installations in addition to Naga Hammadi's 64 MW (42 MW firm dependable) capacity, with the aim of satisfying the peak power requirement by firm plant capacity in the UPS.

## 6.6 EFFECTS OF HYDROPOWER OPERATION ON THE RIVER FLOW CONDITIONS

The most severe effect on steady flow in the river will result from load rejection at the powerplant due to transmission failure or similar. Whilst it must be classified as a rare event, a surge would be created which propagates in both the up- and downstream directions with the wave celerity. The water level immediately upstream at the powerplant would rise instantaneously and downstream the negative surge would cause a sudden drop of water level.

To quantify the transient phenomena of the surge propagation, numerical simulations were undertaken using the hydraulic computer program, CARIMA, which simulates unsteady flow in open channels.

Maximum surge occurs when load rejection is accompanied by the maximum reduction in flow through the turbines. Simulation was therefore performed assuming the maximum turbine discharge of 1,842 $m^3/s$. Immediately following load rejection the flow through the turbines would reduce to about 50% (to 941 $m^3/s$) with the turbines remaining in *sailing mode*. In this mode, the powerplant and sluiceway are operated concurrently. The sluiceway commences to open concurrently with ongoing closure of the turbines until the original river discharge passes totally through the sluiceway. The total time for this operation is approximately 5 minutes.

Figure 6-4 shows the propagation of the surge due to load rejection. The estimated maximum height of the surge (at the powerhouse) is some 0.33m (above Normal Operating Level). This attenuates to less than 0.1m within a distance of 5 km upstream. The simulations did not include the additional attenuation which would occur as the wave passes through the fully opened vents of the existing Barrage. Downstream, the maximum negative surge during load rejection was some 0.31 m at the New Barrage. This attenuates to less than 0.1 m within a distance of 4 km downstream.

# 7. ENVIRONMENTAL IMPACT ASSESSMENT

## 7.1 INTRODUCTION

The construction and operation of the New Barrage is likely to result in a number of environmental impacts, which will be comparable for a Barrage with and without hydropower. Therefore, they are not attributable to the hydropower, but to the irrigation component of the project. The Terms of Reference for the project required that a full Environmental Impact Assessment (EIA) be undertaken to accurately identify and quantify the impacts, develop mitigation and compensation measures for these impacts, and quantify the associated costs. A summary of the EIA, which is presented as a freestanding document comprising a Main Report (Volume 7.1) and associated Annexes (Volume 7.2) of the Feasibility Study Report, is presented below.

## 7.2 RELEVANT NATURAL AND SOCIO-ECONOMIC ENVIRONMENTS

The existing baseline conditions for the project impact area have been determined to provide an overview of the natural and socio-economic conditions, into which the project will be implemented. The collection of these baseline data comprised initial review of project documents and relevant literature and visits on site. A detailed investigation programme was developed based on the initial scoping and discussions were held with the relevant authorities. In response to the findings of the ongoing environmental impact assessment and refinement of the design layout, further additional field surveys and case studies were carried out related to areas in which specific project impacts may result.

The width of the cultivated valley plain in the project area is some 18 km in average, with a narrow point of 13 km at the existing barrage. The upper soil consists of silt and clay providing reasonably fertile conditions for agriculture under natural conditions. This central plain is confined by a mostly uncultivated plain of some 10 m to 25 m higher ground elevation, which is marked by rock mountains of several hundred meters in height.

The water quality in the wider reach up- and downstream of NHB is above standard at present and i characterized by high levels of dissolved oxygen. One excemption is a significant coliforn contamination at Naga Hammadi city, which however is not persistent over the further downstrean reach.

The aquatic micro-flora is considerably divers and several temporarily flooded channels exist whic are colonised with weeds, reeds and floating plants. The flood channels are favourite fish spawnin areas. Due to the existence of the barrages, which prevent upstream fish migration, the population are partly isolated. Of wildlife in the project area, birds represent the most abundant species.

The most recent census data (1994) indicate the population in Qena Governorate is currently of th order of 2.5 million with an overall population density of some 1,480 persons/km$^2$. The number inhabitants in the project area has increased considerably between 1976 and 1986, the annual grow rates over this 10 year period ranging from less than 1% in smaller villages to around 2.5% in larg towns like Abu Tesht and Kous. Some 78% of the population is distributed within the rur community in small villages and hamlets where the population density is 1,300 persons/km Agriculture provides the basis of economic livelihoods for the majority of the population living in t project impact area.

The irrigation network is in a good workable condition and canals are obviously maintained well ensure continuous supply of water. The survey of the existing drainage network howev highlighted significant deficiencies in the maintenance with a large number of the main and second open drains being heavily congested by weeds and water hyacinth. This lack of maintenance of main drains over wide areas is contributing significantly to the relatively high groundwater lev

observed across much of the project area, which are almost independent of the seasonal fluctuation of the headpond level.

Distribution of land holding in the area is based on customary rights which have been brought under statutory law. Plots are measured by cadastral survey and registered in the local land registry, which comes under the jurisdiction of the Ministry of Justice in Qena. Agricultural land, defined as land where water is available for irrigated farming, is delineated by a section and ward of a village. The financial cost of such land is very high and rarely becomes available for sale but is inherited from generation to generation based on the Islamic Sharia system.

Seasonally flooded land under existing conditions, including islands, are Government-owned, with rights of cultivation established by the payment of a land tax. Government land which had previously been subject to seasonal inundation but since commissioning of the HAD is now farmed perennially, has been increasingly rented and leased to farmers.

At present a total of 72% of households are supplied with domestic water either by abstraction from the Nile River normally using package plants located close to the riverbank (9%) or from the deep aquifer using deep wells (63%). These dwellings are primarily located in the main urban and larger rural settlements. The remaining 28%, generally in isolated villages, use handpumps from shallow wells extending into the upper aquifer. There are several programmes under way in the project impact area to connect those using handpumps in the shallow aquifer to a more hygienic water source, aimed at progressively replacing all shallow well handpumps in the villages.

The majority of buildings in the project impact area have only a limited sewage system comprising (i) two unsealed ground tanks, one constructed with a concrete flooring in which the solid waste is collected and the other from bricks through which the liquid waste infiltrates to the upper aquifer (70% of buildings), or (ii) deep cylindrical cesspits constructed of brick walls (25% of buildings) or (iii) isolated septic tanks (5% of buildings). Within the larger towns, the percentage with sealed systems is greater at around 15%.

## 7.3 ENVIRONMENTAL REGULATIONS AND INSTITUTIONAL RESPONSIBILITIES

Prior to developing the Environmental Management Plan (EMP), it was necessary to:

i. Review the existing legislation and regulations governing environmental impact assessment and the compensation process in Egypt.

ii. Compare these regulations with those generally applied by the major lending institutions in similar projects or for the mitigation of similar impacts.

iii. Assess the institutional capabilities of the authorities who would contribute to the implementation of the EMP.

The relevant regulations and laws associated with environmental protection and compensation procedures in Egypt were collated and reviewed by local and foreign experts and compared with directives followed by international funding agencies.

Egyptian Laws and regulations dealing with environmental impacts relevant to the project are primarily those associated with the expropriation and compensation procedures which would have to be implemented by the MOPWWR prior to and during construction. For compensation of privately-owned land and Government-owned land which is leased to farmers, Egyptian Law Nos. 10 and 100 respectively are directly relevant. For privately owned land, the compensation can be either payment for the land at the existing market rate or land-by-land exchange. For Government-owned land no direct compensation for the value of the land is applicable. However, for both land types compensation for lost income must also be paid. For privately-owned land Law no. 10 does not

present guidelines. For government-owned land Law no. 100 specifies its compensation value as '50 times the real estate tax'. No discussion on duration of payment is presented when expropriation is temporary and occurs over a number of years.

The Egyptian regulations were compared with regulations and directives of the international lending agencies such as World Bank Directive 4.00 (Annex A) - Environmental Assessment, and No. 4.30 - Involuntary Resettlement, to ensure all relevant components necessary for the EIA were addressed and to assess any significant disparities with the Egyptian regulations. For those issues relevant to the Naga Hammadi Barrage project, Directive 4.30 outlines the importance of land-by-land settlement while Directive 4.00 discusses the needs of those authorities implementing the EMP having sufficient institutional capability. Both these issues are addressed in the proposed plan, which is in line with the World Bank Directives.

The application of the Egyptian laws no. 10 and no. 100, where the compensation within the EMP is based on, do not result in approaches contrary to the World Bank Directive 4.30.

## 7.4 MAIN ENVIRONMENTAL IMPACTS

For the purposes of the Study, potential impacts were summarised into two main categories, namely construction-related and headpond level-related. The **potential** impacts under these categories are summarised in Table 7.1, together with the assessment of impact direction (positive or negative) and impact severity (negligible, low to moderate, significant) based on results of the subsequent analyses. The kind and extent of impacts are summarized as follows while details are found in Volumes 7.1 and 7.2.

### 7.4.1 Headpond -Level Related Impacts

Headpond-level related impacts will result from the permanent increase in headpond level of the New Barrage compared to the existing seasonal maximums of 65.4 m asl and 65.1 m asl in summer and winter respectively. This increase will result in increased river levels for some distance upstream as discussed in Section 4.1.2. Also groundwater levels are expected to rise due to the increased river level, the effect, however, decreases with the distance to the river (Section 4.2.4). The tailwater area of the existing Barrage will also be inundated over the reach downstream to the New Barrage by a significant depth of some 5 to 6 m compared to the existing situation.

*Land Loss on River Islands and Riverbanks*
The increase in headpond levels will result in the permanent inundation of area of the river islands and riverbanks upstream of the existing Barrage which are currently used for perennial cropping, and seasonal cropping or grazing (currently inundated during summer). In summary, the respective losses are estimated to be 64 feddan, 360 feddan, and 230 feddan respectively.

*Infrastructure Along the River*
The majority of infrastructure along the river has been structurally and operationally designed for a design headpond level of the existing Barrage (65.9 m asl) and for river flows exceeding the present peak flows. No structural protection works will therefore be required as a result of the New Barrage as the headpond level does not exceed the design river levels and anticipated maximum summer flows are lower.

More detailed investigations concerning the operation of open drain outlets, drainage and irrigation pumping stations, and water supply pumping stations were undertaken. The increased river level will result in reductions of pumping energy requirements for the irrigation and water supply pumping stations and an increase for the drainage pump station in El Rawy. In summary, the energy savings are estimated to be 3,712 GWh/y for water supply and irrigation pumping stations while the increased energy requirement for the drainage pumping station is 144 Gwh/y.

## Tabe 7.1: POTENTIAL ENVIRONMENTAL IMPACTS OF PROJECT

Legend
++ : significant positive impact
+ : low to moderate positive impact
no : no impact

- - : significant negative impact
- : low to moderate negative impact
0 : negligible impact

| Environmental Impact / Characteristics of Project | Construction of New Barrage | | | | | | Operation of New Barrage | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Waste Disposal | Dust | Construction Traffic | Temporary Land Requirements | Temporary Labour Requirements | Construction Camp | Headpondlevel | Groundwaterlevel | Permanent Land Requirements Dom Island & Left Bank | Reclaimed Land from Backfill of Floodchannel | Impoundment of Reach between Existing & New Barrage | Hydraulic Characteristics of New Barrage |
| **U/S Reach** | | | | | | | | | | | | |
| *Pysical Impacts* | | | | | | | | | | | | |
| Inundation of agricultural Land in u/s Reach | | | | | | | - - | | | | | |
| Infrastructure along River | | | | | | | | | | | | |
| - roads & railways | | | | | | | no | | | | | |
| - wharfs & industrial plant | | | | | | | no | | | | | |
| - dykes | | | | | | | no | | | | | |
| - drainage pumping stations | | | | | | | no | | | | | |
| - irrigation & water supply pumping stations | | | | | | | no | | | | | |
| - bridges | | | | | | | 0 | | | | | |
| - power & communication lines | | | | | | | no | | | | | |
| - Operation of Irrigation Pumping Stations | | | | | | | + | | | | | |
| - Operation of Drainage Pumping Stations & Outlets | | | | | | | - | | | | | |
| Operation of Domestic Water Supply System | | | | | | | + | no | | | | |
| Agricultural Yield | | | | | | | | - - | | | | |
| Irrigation and Drainage Infrastructure | | | | | | | no | no | | | | |
| Settlements | | | | | | | | | | | | |
| - Buildings | | | | | | | | - - | | | | |
| - Sewage System | | | | | | | | - - | | | | |
| - Domestic Water Supply System | | | | | | | no | no | | | | |
| Historic Sites | | | | | | | no | no | | | | |
| Graveyards | | | | | | | no | 0 | | | | |
| Public Health | | | | | | | no | no | | | | |
| Fisheries (Upstream Reach) | | | | | | | + | | | | - | |
| Habitat Loss | | | | | | | 0 | | | | | |
| *Socio-Economic Impacts* | | | | | | | | | | | | |
| - Disruption of Communities | | | | | | | - 2) | no | | | | |
| - Loss of Income & Livelihood | | | | | | | - - 2) | no | | | | |
| **Vicinity of Construction Site** | | | | | | | | | | | | |
| *Physical Impacts* | | | | | | | | | | | | |
| Balance of agricultural Land at New Barrage | | | | - - | | no | | | - - 1) | ++ 1) | | |
| Relocation of Houses | | | | | | | | | - - | | | |
| Relocation of local Canal & Road | | | | | | | | | - - | | | |
| Irrigation Pumping Facilities on Dom Island | | | | | | | | | - - | | + | |
| Operation of Irrigation Syst. Dom Island & Left Bank | | | | | | | + | | | | ++ | |
| Habitat Loss | | 0 | 0 | 0 | | | | | | | | |
| Naga Hammadi Mini Hydropower Plant | | | | | | | - - | | | | | |
| Fisheries (Flood Channel) | | | | | | | | | | - - | | |
| Health of local Communities | 0 | - | - | | | + | | no | | | | |
| Soil Quality | 0 | | | | | | | | | | | |
| *Socio-Economic Impacts* | | | | | | | | | | | | |
| - Disruption of Communities | | | | - | 0 | | | | 0 | | | |
| - Loss of Income & Livelihood | | | | - - | + | | | | - - 1) | ++ 1) | | |
| - Land Tenure | | | | | | no | | | - | + | | |
| Construction Nuissances (Dust, Noise) | - | - | - | | | | | | | | | |
| **D/S Reach** | | | | | | | | | | | | |
| Irrigation of reclaimed Land d/s | | | | | | | ++ | | | | | |
| Water Quality | 0 | | | | | | | | | | | 0 |
| Fisheries (Downstream Reach) | | | | | | | | | | - | | |
| River Morphology | | | | | | | | | | | | 0 |

1) : Area of reclaimed land exceeds area of lost land
2) : Refers to tenants of inundated land only

Some of the open drain outlets are marginally impounded during peak river discharges under existing conditions. With the increased headpond level, the impoundment will slightly increase. An exception is the outlet of Hammad Drain, which is already considerably impounded under existing conditions, resulting in the necessity of construction of a pumping station even without project. The higher headpond level will contribute to increased dimensions of that pumping station.

Irrigation water for the New Lands of 71 feddan currently under development downstream of Naga Hammadi but outside the boundaries of the existing irrigation network shown on Album no. 84 can be supplied directly via the Eastern and Western canals with a constant headpond level of 65.9 m asl. With a lower level, the discharge capacity of both canals would be insufficient and pumping stations would be required for offtakes directly from the River Nile further downstream. An additional saving in energy of 12,030 GWh/y will result from the provision of water from the Naga Hammadi headpond.

*Impacts of Increased Groundwater Levels in Agricultural Areas*
The groundwater model described above was applied to determine the impacts of increasing the headpond level to a constant value of 65.9 m asl. In undertaking the analyses it was assumed that the planned maintenance of selected reaches of the open drain network, ongoing and committed installation and upgrading programs for subsurface tile drainage by the Drainage Directorate, and a reduction in the irrigation application in the area served by Bachanis Drain (to be in line with that applied over the remainder of the project area) would be implemented by the relevant authorities. On this basis, the **additional** areas over which groundwater levels would increase to within 0.6 m and 0.75 m in winter and summer (critical to associated crop production) would be 975 feddan and 3,625 feddan respectively. Additional areas with a groundwater level increase to within 1.0 m may be critical to fruit gardens, however, all major fruit gardens were found to be located outside of these areas.

*Impacts of Increased Groundwater Levels within Settlements*
Increased groundwater levels are likely to result in both an expansion of the existing problems of wall damp in buildings and houses and additional impacts on sewage systems in the project impact area. For both, a depth to groundwater of 1.0 m was adopted as the minimum acceptable.

The location of additional areas where groundwater levels would rise within that 1 m below surface were compared with the locations of the towns and villages within the project area to identify eventual impacts. This indicated a total of nine villages and towns are either partially or fully located in the additionally affected areas. To quantify the impact on buildings, the population of each town and village was determined from the recent census data (Qena Governorate, 1994). Based on the proportion of the town or village within the additionally affected area and the respective household densities (adopted as 7.5 for smaller villages and 10 for the towns), the overall number of houses affected by damp was estimated to be 4,490.

At present, the predominantly unsealed sewage systems result in a correspondence between sewage and groundwater. This situation is unacceptable in general and is not significantly changed by the level of groundwater as long as unsealed tanks dip into the groundwater at all. However, very shallow groundwater levels may lead to flooding of unsealed septic tanks and in the extreme case to surface pounding of groundwater through capillary rise and to associated offensive odours. Based on the census data, the total affected population in farms and villages was estimated to be 36,500, which includes an allowance for the small percentage of existing sealed tanks within cities.

For domestic water supplies drawing from the upper Holocene aquifer, it is unlikely that small increases in groundwater levels would result in any further deterioration in the water quality. Notwithstanding, several programmes are underway to provide the remaining 28% who currently abstract water from the upper aquifer with a supply from either deep wells or pumping from the Nile River (using compact units). These include the UNICEF-sponsored project in the villages of Farshut District and a Government-sponsored programme in the remainder of the project area. Both will be

completed prior to project implementation. In view of both, the existing poor water quality of the upper holozene aquifer and the water supply improvements underway, the project will not affect domestic water supply.

*Cultural Sites and Graveyards*
The most important cultural site, Dandara Temple, is located some 60 km upstream of the New Barrage where river level increases will only be around 0.2 m during the average summer discharge. In addition, it has a drainage system installed so that no impacts are expected on that site.

Graveyards are traditionally located either in the surrounding hills or on elevated areas in the valley. Generally no impacts are therefore anticipated although more recently some burials have occurred in lower lying areas (although not allowed by the authorities). Increased groundwater levels may affect these locations.

*Fisheries and Habitats*
No significant habitat losses are associated with the increase in headpond level. The inundation of additional areas around the river islands and riverbanks upstream are expected to result in a small increase in fish catch, since these areas will be of shallow water providing large suitable areas for fishing and fish spawning.

*Public Health*
Comparison of the occurrence of typhoid and hepatitis cases in areas of existing high and low groundwater levels the in Esna region before and after commissioning of the New Esna Barrage does not indicate any influence resulting from high groundwater levels for either typhoid or hepatitis. Given these findings and the proposed measures to mitigate against increased groundwater levels, it is considered very unlikely that the project will result in any changes in the existing health situation in the project impact area.

### 7.4.2 Construction-Related Impacts

Construction-related impacts associated with the project are those resulting directly from the construction of a New Barrage. By definition they are those impacts which, unless related to headpond level increases, occur in close proximity to all new structures related to the project. This encompasses the area from the existing Barrage to immediately downstream of the New Barrage including both right and left banks of the Nile River, Dom Island, the flood channel and alignments of the transmission line and new public road connections. The construction-related impacts will be either temporary, extending only over the period of construction, or permanent as a result of a long-term change in the environment.

The significant physical impacts identified are land expropriation (either temporary or permanent), replacement of irrigation pumps, loss of fruit trees, relocation of houses on Dom Island (to a nearby location on the island), and impacts on fish production. The socio-economic impacts are related primarily to loss of income of different categories of people. The actual extent of construction-related impacts differs only marginally for the New Barrage with or without hydropower, the situation is shown on Figure 7.1.

*Land Expropriated and Land Reclaimed*
The actual areas required both permanently and temporarily for both alternatives are quantified in Table 7.2 on the basis of existing and future land utilisation. The principal feature of the project is the temporary loss of perennially cropped land on Dom Island and the left bank which will be replaced (land-by-land) at the end of construction by land reclaimed in the flood channel (and diversion canal) using material excavated during construction. The areas to be used for construction camp and site installation are mainly located on the east bank in unoccupied desert area (Government-owned).

Table 7.2. SUMMARY OF LAND UTILISATION AT THE PROJECT SITE DURING AND AFTER CONSTRUCTION

| Item | New Barrage (feddan) | New Barrage Without Hydropower (feddan) |
|---|---|---|
| Permanently Lost Land for Grazing & Seasonal Cropping | -70.0 | -43.0 |
| Permanently Lost Land for Perennial Cropping | -125.4 | -94.4 |
| Permanently Reclaimed Land for Perennial Cropping | +239.6 | +146.9 |
| Balance of Land for Perennial Cropping | +114.2 | +52.5 |
| Temporarily Occupied Land for Perennial Cropping | -51.2 | -48.9 |

Notes: Perennially cropped land is privately owned
Grazing land and land for seasonal cropping is owned by the government
- Indicates land loss + indicates land gained

Overall, for the New Barrage there will be a permanent expropriation of some 125.4 feddan of perennially cropped land for the project components. However, 229.6 feddan of land in the flood channel which is currently used for seasonal cropping (55 feddan), grazing (10 feddan), or is uncultivated/sandy area unsuitable for agriculture will be infilled and converted to perennial land. This land will be used in land-by-land compensation as discussed in Chapter 6. Further 10 feddan on the west bank will also be reclaimed (upstream section of diversion canal). The extensive reclamation of land results in a positive balance of 114.2 feddan (478,000 $m^2$) land suitable for perennial cropping.

Perennial land expropriated temporarily for construction of the New Barrage is 51.2 feddan. For the construction camp, workshops, and the like, desert land on the east bank will be used.

*Fish Catch*
The principal impacts of the project on fishing will occur in the flood channel where there will be both a significant reduction in fish catch (due to reduced area available for fishing) and a reduction in the spawning areas (resulting in a reduced fish population both in the flood channel itself and also in the downstream reach).

The estimated losses in fish catch during the construction period are estimated to be 42 ton/year. Following project implementation, the fish catch is expected to increase marginally by around 80 ton/year compared to the current catch, due to the increase in catch and spawning areas upstream.

*Loss of Income and Livelihood*
The loss of income and livelihood resulting from the project will affect land owners and tenants, agricultural labourers, fishermen and fish traders, and irrigation pump owners on Dom Island and upstream.

The permanently lost, perennially cropped land on Dom Island and the left bank will be compensated directly on the basis of land-for-land (in accordance with Egyptian Law No. 10) upon completion of construction. However, during the construction period both the landowners (or tenants) and those employed in seasonal and perennial agricultural production will lose income, which is unavoidable, and will therefore be compensated

The reduction of fish catch in the flood channel and the downstream reach to Nak Nak Island during construction will also result in a reduction of income for the fishermen in these areas and indirectly the fishtraders. Since the fish catch in total is expected to increase after construction, it is assumed that a number of fishermen currently operating in the flood channel and downstream of the Barrage would therefore move their operation to the upstream reach.

Due to a reduction in agricultural areas to be irrigated during construction, the income of the pump owners will decrease during the relevant periods. In the longer term, there will actually be a substantial increase in income to these operators following reclamation of the backfill area in the flood channel. In addition the pumping head and therefore energy consumption would also be reduced.

*Social and Cultural Impacts*

At the commencement of construction a total of 40 dwellings on Dom Island (New Barrage with Hydropower) or 24 (New Barrage without Hydropower) will be relocated to new sites within Dom Island. These dwellings are used only for temporary accommodation either overnight or for longer periods during harvesting while their owners have permanent residences on the left bank. Although there will be some social impacts arising, the project **will not require resettlement** of Dom Island residents to new communities on the mainland so that social disruptions of communities will be low. The sites of the reallocations would only be confirmed after in depth discussions with those affected.

With regard to the workforce for construction, which will total some 900 workers at its peak, preference will be given to offering employment to the local population and particularly those whose livelihoods are disrupted during the period of construction. Furthermore, the workforce during construction will open other fields of income like trading or services rendered.

*Health Impacts*

Although no significant impacts are anticipated on the health of the local community as a result of the New Barrage, nor have any been noted in and around Esna since the implementation of the New Esna Barrage, some unavoidable aspects of large construction sites have to be addressed, which are (i) potential cross-contamination by resistant or exotic disease strains between the indigenous and incoming labour forces, (ii) increases in traumatic injuries and (iii) possible increases in road accidents caused by heavy traffic.

## 7.5 SET-UP OF THE ENVIRONMENTAL MANAGEMENT PROGRAMME

On the basis of identified impacts, appropriate mitigation, compensation or preventive measures have been developed and fully costed in the form of an Environmental Management Plan. A comprehensive Environmental Monitoring Plan has also been prepared aimed at ensuring the mitigation and compensation measures can be successfully implemented and will fully meet the objectives.

In evaluating the mitigation or compensation measures and costs, the principles applied were:

(i) Mitigation measures should be given preference over (cash) compensation.

(ii) Affected material assets shall be replaced by an appropriate substitute.

(iii) Compensation payments shall be made only to those directly affected.

(iv) Owners of permanently expropriated private land should be compensated with land of the same economic value wherever possible (land-by-land compensation). Only the lost income during the time span between expropriation and return of land and initially reduced yields of this land shall be compensated by cash. In determining the land-by-land compensation, every effort should be made to consolidate the individual land tenure which at present is often fragmented through inheritance, etc.

(v) Owners of temporarily expropriated private land will be compensated in cash during its occupation.

(vi) Tenants of permanently used government-owned land will be compensated for loss of crop production for two years.

(vii) Agricultural labour (on perennially cropped land) and fishermen/fish traders will receive indemnity for lost income for a maximum of either two years or in the case of agricultural labour the duration of temporary land expropriation, whichever is less.

(viii) Rates of compensation payment during occupation of agricultural land and for lost income from fisheries are based on the respective net returns or incomes.

(ix) Rates of indemnity for lost income of agricultural labour are based on full labour wages.

(x) Cash payments would be made on an annual basis.

Table 7.3: SUMMARY OF ENVIRONMENTAL MITIGATION AND COMPENSATION PLAN

| Impact and Location | Mitigation/Compensation | Cost $10^6$ LE | |
|---|---|---|---|
| | | With Hydropower | Without Hydropower |
| Perennially cropped land lost permanently - Dom Island and left bank | Land-by-land compensation<br>Cash compensation for lost agricultural production during onstruction and reduced yields for two years | 0.0<br>2.595 | 0.0<br>1.979 |
| Perennially cropped land lost permanently -left bank | Cash compensation for small areas used for transmission towers | 0.112 | 0.0 |
| Seasonally cropped or grazing land lost permanently - Dom island, flood channel, U/S river islands and riverbanks | Two years compensation for lost agricultural production. | 0.971 | 0.927 |
| Perennially cropped land lost permanently - U/S river islands | Government owned - two years compensation for lost production | 0.280 | 0.280 |
| Perennially cropped land lost temporarily - Dom Island and left bank | Cash compensation for period of construction or expropriation | 0.769 | 0.747 |
| Lost income for labourers on perennially cropped land | Indemnity for two years labour | 0.279 | 0.237 |
| Reduction in fish catch | Indemnity for two years lost income based on reduced catch (including fishermen and fishtraders) | 0.328 | 0.328 |
| Increased dimension of new Hammad Drain Pumping Station | Cost Share 5% | 0.557 | 0.557 |
| Temporary reduction in irrigation pumping requirements on Dom Island | Indemnity for lost income for period of reduced pumping | 0.144 | 0.078 |
| Increased groundwater in additionally affected agricultural area | Installation of subsurface tile drainage | 2.228 | 2.228 |
| Effects on buildings from increased groundwater | Installation of asphalt sealing or metal sheeting layer above building foundations and below ground floor to damp proof houses at risk | 6.735 | 6.735 |
| Effects on septic ranks from increased groundwater | Construction of sealed septic tanks with pipe drainage network and provision of additional trucks to transport effluent to desert. | 3.910 | 3.910 |
| Construction required in locations of material assets - canal, houses on Dom Island, service road and canal left bank, mosque, irrigation pumps and banana plantation on Dom Island | Relocation/replacement of material assets | 1.152 | 0.824 |
| Total | | 20.059 | 18.829 |

Cost estimates for replacement of material assets such as houses, canals or roads, and for mitigation measures including installation of tile drains or septic tanks are based on the relevant construction cost estimates at a 1995 cost level. Lost agricultural and fish production were calculated on the gross margin. The procedures applied for compensation were determined after considering Egyptian Law nos. 10 and 100 outlining respectively the compensation for expropriation of private and Government land (leased to farmers). In addition, the methods applied in recent compensation schemes in the

project area were also considered prior to adopting specific rates of compensation. The methods of mitigation and compensation and associated costs are briefly summarised in Table 7.3.

## 7.6 INSTITUTIONAL INVOLVEMENT

The overall coordination of environmental regulations is ultimately the responsibility of the Egyptian Environmental Affairs Agency (EEAA). However, in implementing the regulations specifically relating to the relevant environmental regulations and Laws, which will be used as the basis for compensating those affected by the project, other various Governmental departments are directly involved.

The MOPWWR, as the proponent of the project, will play the pivotal role. Given the stated intention of the Egyptian Environmental Affairs Agency (EEAA) that the implementation of the Environmental Management Plan is the responsibility of the proponent and the fact that the capabilities of the MOPWWR in this area are at present extremely limited, the Ministry has declared its intention to establish an Environmental Unit (EU). The Unit, funded as part of the overall Environmental Management Plan, would have sufficient resources and institutional capacity to:

- Control and oversee implementation of the mitigation and compensation procedures through liaison with the various authorities directly involved, but having ultimate responsibility for their success.

- Undertake collection of baseline data and ongoing data measurement, research and collaborative studies, and monitoring of programme success eg drainage assessment, groundwater response.

- Provide long-term institutional strengthening of the MOPWWR to enable it to deal effectively with environmental management, public participation and effective coordination with other organisations .

The set-up of the Environmental Unit with personal from Egyptian institutions including employment of Egyptian and foreign specialist staff is discussed in detail in Volume 7.1, Section 6.6.

## 7.7 MITIGATION AND RESIDUAL IMPACTS

Despite the implementation of the EMP (Table 7.3), a number of residual negative impacts are likely as a result of the project for which mitigation or compensation measures are not feasible. The residual impacts upon implementation of the EMP are summarised in Table 7.4, which also includes the desired positive impacts. Residual impacts can be temporary or permanent.

The temporary residual impacts comprise the temporary physical land requirements and the construction traffic, which are the usual unavoidable effects of large construction sites. The loss of income however, resulting from the temporary physical land requirement will be fully compensated in cash, so that no economic impact on the local community arises.

The mitigation of further potential nuisances that may result from waste disposal, dust development, etc. will be within the responsibilities of the Contractor and its successful implementation monitored by the EU.

Permanent residual impacts in the vicinity of the construction site are the positive balance between land consumption and land reclamation with its positive permanent impact on the local agricultural production and job creation. However, this results in the reduction of fish yield in the flood channel and in the downstream reach. It will be the task of the EU to implement measures for local improvements in fishery as discussed Volume 7.1, in section 6.

Table 7.4: SUMMARY OF PROJECT IMPACTS AFTER MITIGATION

Legend
++ : significant positive impact
+ : low to moderate positive impact
no : no impact

-- : significant negative impact
- : low to moderate negative impact
0 : negligible impact

☐ : direction and/or severity of impact changed with Environmental Management Plan

| Characteristics of Project | Construction of New Barrage | | | | | | Operation of New Barrage | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Environmental Impact | Waste Disposal | Dust | Construction Traffic | Temporary Land Requirements | Temporary Labour Requirements | Construction Camp | Headpondlevel | Groundwaterlevel | Permanent Land Requirements Dom Island & Left Bank | Reclaimed Land from Backfill of Floodchannel | Impoundment of Reach between Existing & New Barrage | Hydraulic Characteristics of New Barrage |
| **U/S Reach** | | | | | | | | | | | | |
| *Physical Impacts* | | | | | | | | | | | | |
| Inundation of agricultural Land in u/s Reach | | | | | | | -- | | | | | |
| Infrastructure along River | | | | | | | | | | | | |
| - roads & railways | | | | | | | no | | | | | |
| - wharfs & industrial plant | | | | | | | no | | | | | |
| - dykes | | | | | | | no | | | | | |
| - drainage pumping stations | | | | | | | no | | | | | |
| - irrigation & water supply pumping stations | | | | | | | no | | | | | |
| - bridges | | | | | | | 0 | | | | | |
| - power & communication lines | | | | | | | no | | | | | |
| - Operation of Irrigation Pumping Stations | | | | | | | + | | | | | |
| - Operation of Drainage Pumping Stations & Outlets | | | | | | | 0 | | | | | |
| Operation of Domestic Water Supply System | | | | | | | + | no | | | | |
| Agricultural Yield | | | | | | | | 0 | | | | |
| Irrigation and Drainage Infrastructure | | | | | | | no | no | | | | |
| Settlements | | | | | | | | | | | | |
| - Buildings | | | | | | | | 0 | | | | |
| - Sewage System | | | | | | | | + | | | | |
| - Domestic Water Supply System | | | | | | | no | no | | | | |
| Historic Sites | | | | | | | no | no | | | | |
| Graveyards | | | | | | | 0 | 0 | | | | |
| Public Health | | | | | | | 0 | 0 | | | | |
| Fisheries (Upstream Reach) | | | | | | | + | | | | - | |
| Habitat Loss | | | | | | | 0 | | | | | |
| *Socio-Economic Impacts* | | | | | | | | | | | | |
| - Disruption of Communities | | | | | | | 0 | no | | | | |
| - Loss of Income & Livelihood | | | | | | | no | no | | | | |
| **Vicinity of Construction Site** | | | | | | | | | | | | |
| *Physical Impacts* | | | | | | | | | | | | |
| Balance of agricultural Land at New Barrage | | | | -- | | no | | | ++ | | | |
| Relocation of Houses | | | | | | | | | no | | | |
| Relocation of local Canal & Road | | | | | | | | | no | | | |
| Irrigation Pumping Facilities on Dom Island | | | | | | | | | no | | + | |
| Operation of irrigation Syst. Dom Island & Left Bank | | | | | | | + | | | | ++ | |
| Habitat Loss | | 0 | 0 | 0 | | | | | | | | |
| Naga Hammadi Mini Hydropower Plant | | | | | | | -- | | | | | |
| Fisheries (Flood Channel) | | | | | | | | | | - | | |
| Health of local Communities | no | 0 | - | | | + | | no | | | | |
| Soil Quality | no | | | | | | | | | | | |
| *Socio-Economic Impacts* | | | | | | | | | | | | |
| - Disruption of Communities | | | | 0 | 0 | | | | 0 | | | |
| - Loss of Income & Livelihood | | | | no | + | | | | no | | | |
| - Land Tenure | | | | | | no | | | + | | | |
| Construction Nuissances (Dust, Noise) | no | 0 | - | | | | | | | | | |
| **D/S Reach** | | | | | | | | | | | | |
| Irrigation of reclaimed Land d/s | | | | | | | ++ | | | | | |
| Water Quality | 0 | | | | | | | | | | | 0 |
| Fisheries (Downstream Reach) | | | | | | | | | | - | | |
| River Morphology | | | | | | | | | | | | 0 |

The cease of the Naga Hammadi Mini Hydropower plant is unavoidable, however, the loss of some 39 GWh/year of energy generation is more than offset by the New Barrage power plant with a generation of 462 GWh/year.

The irrigation of reclaimed areas downstream of the New Barrage will be possible via the existing main irrigation canals instead of pumping from the tailwater and hence resulting in a considerable permanent benefit.

In the upstream reach, the physical inundation of agricultural land cannot be avoided. None of this land is privately owned and most of it consists of grazing land and land for seasonal cropping of low agricultural value. The current tenants and agricultural labour will be compensated in cash for loss of income for a period of two years. As a result of the inundation, attractive fish spawning areas increase due to the more shallow river banks.

The predicted groundwater levels will nowhere rise as high as to affect the yield of the perennial sugar cane, which is the main crop in the area. Potential impacts on the agricultural yield of seasonal crops due to rising groundwater levels will be mitigated by installation of subsurface drains in the areas affected by the project. An exemption are small and scattered additionally affected areas at Hammad Drain (East) and Shanhoria Drain, which are distributed as a thin ribbon around the exterior of those areas affected already under existing conditions because there, the implementation of a mitigation programme involving tile drains is not realistic. These areas with seasonal crops in the district of Deshna amount to some 525 feddan. It must be noted, however, that the predicted rise of groundwater levels in this area in real terms is below 10 cm and thus results in a low residual impact.

Potential impacts of rised groundwater level on settlements were predicted in form of damp that may affect buildings and flooding of unsealed cess pits. Mitigation measures were therefore foreseen to seal affected houses against damp and to replace the existing cess pits by sealed septic tanks in the affected settlement areas. The replacement of unsealed cess pits will in general improve the sanitation conditions irrespective of the project.

The studies and ongoing adaptation of the EMP during and after the construction of the project may well result in some of the residual impacts becoming negligible or mitigation plans being developed. For example the loss of agricultural employment may be offset by employment in the actual construction. Therefore every effort must be made to ensure successful implementation and operation of the EU.

## 7.8 UNCERTAINTIES AND RISK

The proposed Environmental Management Plan and Monitoring Programme remain subject to uncertainty and risk due to:

i. Inaccuracies in data available for the study.
ii. Limitations in methods to quantify impacts.
iii. General uncertainties on specific issues for which data were simply not available.

These issues, which are outlined below, can to a large extent be removed or minimised through ongoing environmental studies during the project's detailed design stage and also the period of construction. The principal areas in which some uncertainties remain are:

- Sustainability of the drainage maintenance programme.
- Impacts of high groundwater levels on agricultural production.
- Impacts of construction and flood channel reclamation on fisheries and fish spawning.
- Crop yields of reclaimed land.

## 7 9 CONCLUSION FROM THE ENVIRONMENTAL IMPACT ASSESSMENT

Decision-making to proceed with the project requires evaluation of the residual impacts as a whole versus the original purpose of the project. The primary function of the New Barrage is to provide long term the headpond for irrigation of some 752 feddan (3,206 $km^2$) of agricultural land, which is the basis of economic livelihood for the majority of population of two governorates totalling in more than 4 million people. This purpose is therefore of national importance to the Arab Republic of Egypt. In addition, the hydropower component will generate the annual energy of 462 Gwh/year. Considering the disasterous consequences of doing nothing and remembering that the Environmental Management Plan set up within the EIA provides for mitigation of the important environmental impacts, it is recommended that the MOPWWR and EEA/HPPEA proceed with the project. Thereby, the unavoidable residual impacts should be accepted in order to long term maintain irrigation in the areas depending on the barrage at Naga Hammadi and to obtain the benefits of energy generation by hydropower without production of carbon dioxide ($CO_2$) emissions.

# 8. PROJECT IMPLEMENTATION

## 8.1 GENERAL

The main purpose of this Chapter is to propose a tentative implementation schedule for the project which demonstrates the different activities to be followed by the implementing institutions, together with suggestions on the organization required to bring the project foreword to financing, environmental acceptance, contracting, construction and commissioning. In this regard, it provides an amalgamation of the issues already discussed with the MOPWWR, the HPPEA/EEA and the KfW, and suggestions made by the NHBD Consultants in their function as consultants to the MOPWWR.

By agreement of the MOPWWR and the MEE from 1996, both the RGB/MOPWWR and HPPEA/EEA have formed a Steering Committee with the aim to jointly proceed the implementation of the New Barrage. It is foreseen, that the Steering Committee will appoint a Project Implementation Unit.

An additional aim of this Chapter is also to summarize the financial cost of the Naga Hammadi Barrage Development, assigning the relevant cost components as appropriate to the MOPWWR and the MEE. This split of investment costs for the different sectors as well as different lots considers the Minister's agreement between the MOPWWR and MEE and the findings dewing the KfW project appraisal mission (section 8.2). This split of investment cost has been used in the financial analysis described in Chapter 10.

## 8.2 IMPLEMENTATION SCHEDULE

After submittal of the draft Feasibility Study, at the end of February 1997, the report was reviewed by the RGB/MOPWWR and HPPEA/EEA, the Panel of Experts (POE) and relevant Authorities and Institutions involved. As a result of the review and a subsequent meeting of the Panel of Experts, the POE submitted a report at the end of April, assessing the project as technically sound and feasible. Subsequently, an appraisal mission of KfW took place in mid-May 1997, resulting in an Aide Memoire signed by the MOPWWR, HPPEA, EEA and KfW with major finding as follows:

- MOPWWR, HPPEA and EEA are in agreement with the results and recommendations of the study, only EEA requested the transmission line voltage to be 220 kV instead of the proposed 132 kV, although the latter voltage would meet the project requirements.

- It was agreed that the NHBD Consultant and the HRI shall proceed with the Hydraulic Model Tests as described in Appendix L6 (and which are currently underway).

- An Environmental Group will be set up in the MOPWWR. In June 1997, the RGB has started to establish the Environmental Group.

- The further engineering services for the project - tender design, documents and evaluation, preparation of contract documents, construction design and supervision - will be performed according to FIDIC rules part 1and 2.

- Principal agreement has been reached that KfW will finance the foreign cost of all electrical mechanical and hydromechanical equipment, the environmental cost, the cost of engineering and the POE and a minor part of the civil works. Prior to definite financing agreements becoming effective the Enviromental Group will be financed under the RGB budget.

- The construction contract shall be tendered in five different lots as reflected in Table 8.2.

A suggested sequence of further proceeding, which is shown in more detail on Figure 8-1, and which would lead to tendering of the project and award of the construction contract in the first half of the year 2000 involves the following further steps:

(i) Following the agreement with KfW to finance the Engineering Services, the tender design, engineering report and preparation of tender documentation could start in mid-1998. This would overlap with the hydraulic model tests currently underway which are expected to be finalised in August 1998.

(ii) Pre-qualification of contractors should take place during the preparation of tender documentation.

(iii) Simultaneous to the loan agreements for foreign currency financing, the agreement to finance the local currency portions must be made by the relevant Egyptian Institutions. This would also include components of the environmental mitigation prgramme.

(iv) The RGB must proceed in establishing the Environmental Unit, to be fully in service from 1998.

(v) The documentation prepared by the Consultant and the pre-qualification results will require approval by the PIU and, if required, by the financiers.

(vi) The PIU can then, with the approved tender documentation, proceed to tendering. The tender phase will be terminated by the evaluation of tenders and recommendation by the Consultant on the ranking of the proposals.

(vii) The PIU will, after approval of the ranking, invite the successful tenderers for contract negotiations. The total contract amounts may then be subject to approval of Government authorities and the financiers before the final awarding of contracts is possible.

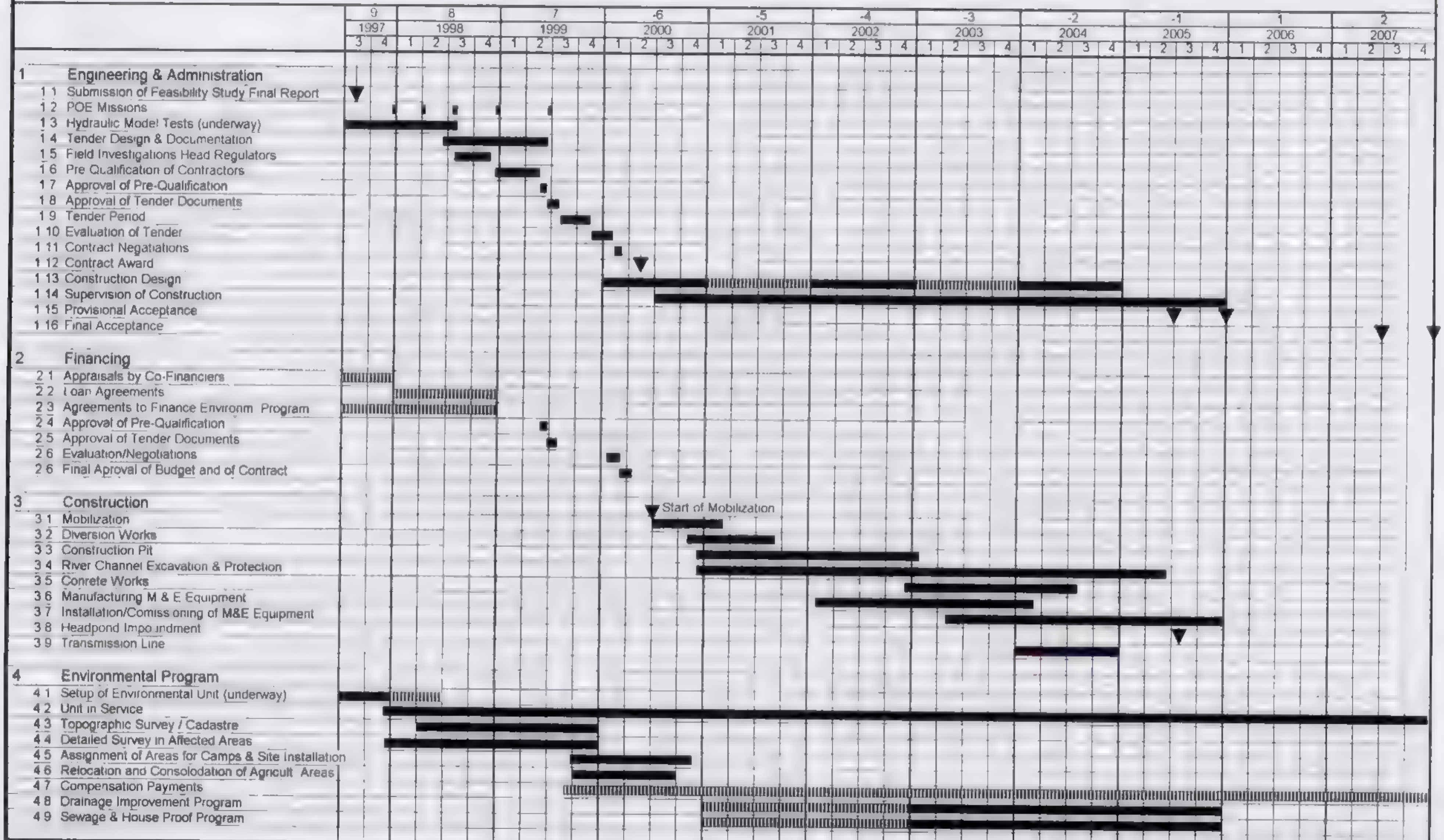
(viii) Commencement of mobilization and construction will need to be at an appropriate time, for example in June, 2000 so that river diversion falls within the dry season in 2001.

(ix) The further tentative programme of project implementation associated with the construction phase can be seen in Figure 8-1.

It is of importance that during the tender design phase the Environmental Unit is fully equipped with the personnel and logistic resources and is able to fulfil the tasks assigned as proposed in Chapter 7. Through this unit, the necessary forums to establish a collaborative approach with the local communities affected either through relocation or temporary expropriation of land required for construction activities and site access would be created. In addition, the unit must also commence planning and implementation of the numerous investigative and monitoring programmes related to environmental issues which form the Environmental Management Plan, as also outlined in Chapter 7. In particular, cadastral and health surveys, groundwater observations, and land expropriation must commence well in advance of construction to ensure environmental mitigation measures (currently proposed) are efficiently implemented and applied as an ongoing part of EMP. Thereafter, the areas required for the contractor's camp and site establishment will have to be cleared and demarcated.

Figure 8-1 shows a tentative schedule of the main tasks of engineering, financing, construction, and the environmental management programme both in advance and during construction of the project up to its implementation. The Schedule avoids details of the tender procedure and of the award of contract including the assurance of funds for the project, as these issues depend on the conditions of the different financiers and the procedures applied by the Egyptian authorities involved.

**Figure 8.1: Tentative Program for Project Implementation**




IMPLEM XLS / GH / 11 08 1997

## 8.3 IMPLEMENTING INSTITUTIONS

The Ministry of Public Works and Water Resources (MOPWWR) and the Ministry of Electricity and Energy (MEE) are involved in the New Barrage Project at Naga Hammadi. Organization charts of both institutions are given in Figures 8-2 and 8-3.

The MOPWWR is responsible for all works related to the management of water resources in Egypt, and for all planning and construction of hydraulic structures and other infrastructural works necessary to fulfil its objectives. Most important within the Ministry are the Irrigation Department, having almost the status of an Authority, the Reservoirs and Grand Barrages Sector, which is responsible for construction and maintenance of all barrages on the River Nile, and the Drainage Authority. Other activities of the Ministry include water regulation, land reclamation and navigation.

The Ministry of Electricity and Energy (MEE) is organized into seven main Authorities, of which two are involved in the New Barrage Project:

- HPPEA (Hydropower Project Executing Authority) is responsible for the design and construction of the power component of the New Barrage (see Figure 8-4, organization chart),

- EEA (Egyptian Electricity Authority) will be the future owner and operator. EEA is responsible for generation and transmission of almost all electricity in Egypt (see Appendix Y, Figure Y-1, organization chart).

In the present organizational framework for the Naga Hammadi Barrage Project, the MOPWWR and the HPPEA/EEA are jointly responsible for planning, financing and implementation of the New Barrage and the supervision of the PIU. MOPWWR will cooperate with the HPPEA in the construction of the civil works of the hydropower component.

## 8.4 SHARE OF PROJECT COST AND FINANCING

The total Construction Costs of the New Barrage are estimated in Appendix T and summarized in Chapter 4, Table 4.1. By agreement of the two Ministries involved by their organizations in the Steering Committee of the project, the HPPEA/EEA will assume the cost of the mechanical and electrical installations of the hydropower plant. The MOPWWR/RGB will assume the remainder of the costs of the project, e.g. the cost of all civil works of the New Barrage and remaining installations, the cost for environmental mitigation, and the cost for engineering up to commissioning. These are summarized in Table 8.1

Table 8.1: SHARE OF CONSTRUCTION COST

| Purpose | MOPWWR $10^6$ US$ | HPPEA/EEA $10^6$ US$ |
|---|---|---|
| Barrage Works (Lots 1 and 2) | 199.6 | |
| Engineering | 28.0 | |
| Generation, Mechanical and Electrical Equipmen. (Lots 3, 4 and 5) | | 95.2 |
| Environmental Mitigation | 13.8 | |
| Total | 241.4 | 95.2 |

A tentative Cashflow of the Investment Costs for the project is given in Table 8.2, with a tentative split into local and foreign currencies and also considering the separation of cost items into procurement lots as proposed during the KfW project appraisal mission.

## Table 8.2 : NEW BARRAGE - SHARE OF CONSTRUCTION COST – CASH FLOW

Fuel = Local

| No | Item of Works or Cost | Amount Million US$ | | | -8 | | -7 | | 6 | | 5 | | -4 | | 3 | | 2 | | 1 | | 1 | | 2 | | 3-24 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 2000 | | 2001 | | 2000 | | 2001 | | 2002 | | 2003 | | 2004 | | 2005 | | 2006 | | 2007 | | 2008 2029 | |
| | | Local | Foreign | Total | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreig | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign | Local | Foreign |
| **MOPWWR** | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | **Lot 1: Civil Works** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Site installation | 8.49 | 2.11 | 10.60 | | | | | 4.67 | 1.16 | 0.85 | 0.21 | 0.85 | 0.21 | 0.85 | 0.21 | 0.85 | 0.21 | 0.42 | 0.11 | | | | | | |
| 2 | Existing Barrage Works | 1.05 | 0.39 | 1.44 | | | | | | | 0.53 | 0.19 | 0.53 | 0.19 | | | | | | | | | | | | |
| 3 | Diversion Canal | 4.18 | 11.97 | 16.16 | | | | | 0.84 | 2.39 | 2.51 | 7.18 | | | | | | | 0.84 | 2.39 | | | | | | |
| 4.1 | Construction Pit – Construction | 10.84 | 18.20 | 29.04 | | | | | | | 4.82 | 8.09 | 4.82 | 8.09 | | | | | 1.20 | 2.02 | | | | | | |
| 4.2 | Construction Pit – Dewatering | 0.40 | 1.62 | 2.03 | | | | | | | | | 0.05 | 0.20 | 0.20 | 0.81 | 0.15 | 0.61 | | | | | | | | |
| 5 | River Channel – Exc. & Protection | 5.74 | 18.87 | 24.60 | | | | | 0.48 | 1.57 | | | 0.96 | 3.14 | 1.43 | 4.72 | 1.91 | 6.29 | 0.96 | 3.14 | | | | | | |
| 6 | Right Bank Closure Dyke | 0.73 | 1.21 | 1.95 | | | | | 0.37 | 0.61 | | | 0.37 | 0.61 | | | | | | | | | | | | |
| 7 | Left Bank Protection Dyke | 0.86 | 1.04 | 1.90 | | | | | | | 0.86 | 1.04 | | | | | | | | | | | | | | |
| 8 | Closure Dam | 2.30 | 3.95 | 6.25 | | | | | | | | | | | | | | | 2.30 | 3.95 | | | | | | |
| 9 | Navigation Lock incl. Guide Walls | 8.40 | 7.52 | 15.92 | | | | | | | | | | | 5.60 | 5.01 | 2.80 | 2.51 | | | | | | | | |
| 10 | Sluiceway incl. End Abutment | 6.34 | 5.58 | 11.92 | | | | | | | | | | | 4.23 | 3.72 | 2.11 | 1.86 | | | | | | | | |
| 11 | Powerhouse incl. Piers | 12.25 | 10.77 | 23.01 | | | | | | | | | | | 8.16 | 7.18 | 4.08 | 3.59 | | | | | | | | |
| 12 | Service Buildings & Switchyard | 0.68 | 0.83 | 1.50 | | | | | | | | | | | | | | | 0.68 | 0.83 | | | | | | |
| | **Subtotal Lot 1** | **62.26** | **84.06** | **146.32** | | | | | | | | | | | | | | | | | | | | | | |
| | **Lot 2: Hydro-mechanical Equipment** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Sluiceway | 3.71 | 9.64 | 13.36 | | | | | | | 0.37 | 0.96 | 0.74 | 1.93 | 1.34 | 3.47 | 0.89 | 2.31 | 0.37 | 0.96 | | | | | | |
| 2 | Navigation Lock | 1.49 | 3.04 | 4.53 | | | | | | | | | 0.15 | 0.30 | 0.30 | 0.61 | 1.04 | 2.13 | | | | | | | | |
| 3 | Irrigation Head Regulators | 1.06 | 0.71 | 1.77 | | | | | | | | | | | | | 1.06 | 0.71 | | | | | | | | |
| 4 | Powerhouse | 2.25 | 6.87 | 9.13 | | | | | | | | 0.69 | | 1.37 | | 2.29 | 2.25 | 2.52 | | | | | | | | |
| | **Subtotal Lot 2** | **8.52** | **20.26** | **28.79** | | | | | | | | | | | | | | | | | | | | | | |
| | **Basic Construction Cost** | **70.78** | **104.32** | **175.10** | | | | | **6.35** | **5.73** | **9.93** | **18.37** | **8.46** | **16.05** | **22.11** | **28.02** | **17.16** | **22.73** | **6.77** | **13.41** | | | | | | |
| | **Contingencies** | 9.68 | 14.84 | 24.53 | | | | | 0.95 | 0.86 | 1.74 | 3.09 | 1.50 | 2.67 | 2.37 | 3.21 | 1.99 | 2.79 | 1.14 | 2.22 | | | | | | |
| | **Total Construction Cost** | **80.46** | **119.17** | **199.63** | | | | | **7.30** | **6.60** | **11.67** | **21.46** | **9.96** | **18.73** | **24.48** | **31.23** | **19.15** | **25.52** | **7.91** | **15.63** | | | | | | |
| | **Engineering for Total Project** | **5.60** | **22.41** | **28.01** | | | | | **0.20** | **1.12** | **0.47** | **2.68** | **0.52** | **2.92** | **1.21** | **6.84** | **1.34** | **7.61** | **0.47** | **2.64** | | | | | | |
| | **Environmental Cost** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Compensation for lost Land | 0.80 | | 0.80 | | | | | 0.11 | | 0.11 | | 0.11 | | 0.11 | | 0.15 | | 0.11 | | 0.05 | | 0.02 | | | |
| 2 | Compensation for lost Income | 0.81 | | 0.81 | | | | | 0.14 | | 0.14 | | 0.04 | | 0.05 | | 0.05 | | 0.22 | | 0.18 | | | | | |
| 3 | Hammad Drain Pumping Station | 0.16 | | 0.16 | | | | | | | | | | | | | 0.16 | | | | | | | | | |
| 4 | Installation/Maint. of Tile Drains | 0.66 | | 0.66 | | | | | | | | | | | | | 0.49 | | 0.01 | | 0.01 | | 0.01 | | 0.14 | |
| 5 | Damp Proof of Houses | 1.98 | | 1.98 | | | | | | | | | | | | | 1.98 | | | | | | | | | |
| 6 | Installation of Septic Tanks | 1.15 | | 1.15 | | | | | | | | | | | | | 1.15 | | | | | | | | | |
| 7 | Replacement of Material Assets | 0.34 | | 0.34 | | | | | 0.34 | | | | | | | | | | | | | | | | | |
| 8 | Environmental Unit | 2.60 | 3.48 | 6.08 | 0.49 | 0.48 | 0.20 | 0.32 | 0.26 | 0.35 | 0.25 | 0.35 | 0.25 | 0.35 | 0.25 | 0.41 | 0.25 | 0.35 | 0.25 | 0.35 | 0.20 | 0.29 | 0.20 | 0.29 | | |
| | **Basic Environmental Cost** | **8.50** | **3.48** | **11.97** | **0.49** | **0.48** | **0.20** | **0.32** | **0.86** | **0.35** | **0.51** | **0.35** | **0.40** | **0.35** | **0.41** | **0.41** | **4.23** | **0.35** | **0.59** | **0.35** | **0.43** | **0.29** | **0.23** | **0.29** | **0.14** | |
| | **Contingencies** | 1.27 | 0.52 | 1.80 | 0.07 | 0.07 | 0.03 | 0.05 | 0.13 | 0.05 | 0.08 | 0.05 | 0.06 | 0.05 | 0.06 | 0.06 | 0.63 | 0.05 | 0.09 | 0.05 | 0.06 | 0.04 | 0.04 | 0.04 | 0.02 | |
| | **Total Environmental Cost** | **9.77** | **4.00** | **13.77** | **0.57** | **0.55** | **0.22** | **0.36** | **0.98** | **0.40** | **0.58** | **0.40** | **0.46** | **0.40** | **0.47** | **0.47** | **4.86** | **0.40** | **0.68** | **0.40** | **0.50** | **0.33** | **0.27** | **0.33** | **0.17** | |
| | **TOTAL COST MOPWWR** | **95.84** | **145.58** | **241.41** | **0.57** | **0.55** | **0.22** | **0.36** | **8.49** | **8.12** | **12.72** | **24.53** | **10.93** | **22.04** | **26.15** | **38.54** | **25.35** | **33.54** | **9.05** | **18.67** | **0.50** | **0.33** | **0.27** | **0.33** | **0.17** | |
| **HPPEA / EEA** | | | | | | | | | | | | | | | | | | | | | | | | | | |
| | **Lot 3: Generation Equipment** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Bulb Turbines | 3.93 | 27.52 | 31.45 | | | | | | | | | | 2.75 | | 14.68 | 3.93 | 7.34 | | 2.75 | | | | | | |
| 2 | Generators incl. Excitation | 3.20 | 18.11 | 21.30 | | | | | | | | | | 1.81 | | 3.62 | 3.20 | 10.86 | | 1.81 | | | | | | |
| | **Subtotal Lot 3** | **7.13** | **45.62** | **52.75** | | | | | | | | | | | | | | | | | | | | | | |
| | **Lot 4: Mechanical & Electrical Equipment** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | Mechanical Auxiliary Systems | 0.64 | 4.48 | 5.12 | | | | | | | | | | 0.45 | | 2.24 | 0.64 | 1.34 | | 0.45 | | | | | | |
| 2 | Electrical Equipment | 4.66 | 8.65 | 13.30 | | | | | | | | | | 0.86 | | 4.03 | 3.72 | 2.31 | 0.93 | 1.44 | | | | | | |
| 3 | Transformers and Switchgears | 3.34 | 8.16 | 11.50 | | | | | | | | | | 0.83 | | 1.66 | 3.34 | 4.85 | | 0.83 | | | | | | |
| | **Subtotal Lot 4** | **8.64** | **21.29** | **29.92** | | | | | | | | | | | | | | | | | | | | | | |
| | **Lot 5: Transmission Line** | | | | | | | | | | | | | | | | | | | | | | | | | |
| 1 | 220 kV Transmission line | 3.11 | 0.78 | 3.89 | | | | | | | | | | 0.08 | | 0.16 | 3.11 | 0.47 | | 0.08 | | | | | | |
| | **Subtotal Lot 5** | **3.11** | **0.78** | **3.89** | | | | | | | | | | | | | | | | | | | | | | |
| | **Basic Construction Cost** | **18.87** | **67.69** | **86.56** | | | | | | | | | | **6.78** | | **26.39** | **17.94** | **27.17** | **0.93** | **7.35** | | | | | | |
| | **Contingencies** | 1.89 | 6.77 | 8.66 | | | | | | | | | | 0.68 | | 2.64 | 1.79 | 2.72 | 0.09 | 0.74 | | | | | | |
| | **TOTAL COST HPPEA / EEA** | **20.75** | **74.46** | **95.21** | | | | | | | | | | **7.46** | | **29.02** | **19.73** | **29.89** | **1.02** | **8.09** | | | | | | |

## 8.5 JOINT ORGANIZATIONAL SET-UP FOR PROJECT IMPLEMENTATION

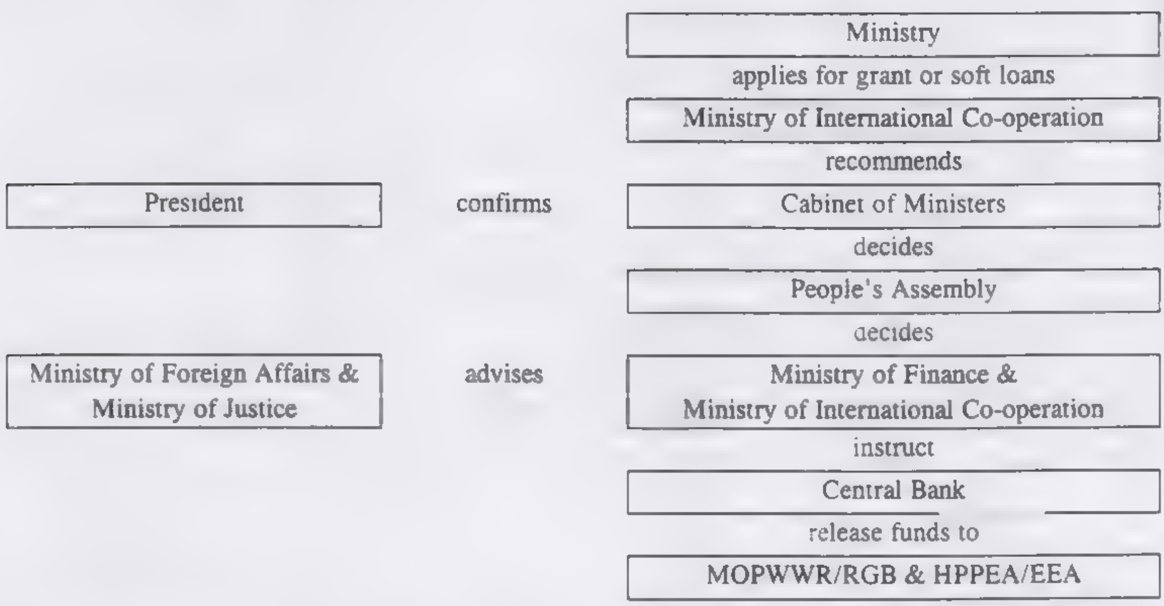
Without referring to details which still need to be agreed between the parties involved, the following organisational set-up is envisaged for the Naga Hammadi Barrage Development:

- Joint project co-ordination is undertaken by a Steering Committee.
- Investment is channelled through the MOPWWR and HPPEA/EEA.
- The MOPWWR/RGB, under the supervision of the PIU, is responsible for the construction of the water retaining structure including the primary concrete works and steel structural installations of the hydropower plant.
- The HPPEA, under the supervision of the PIU, is responsible for the installation of the generation, mechanical and electrical equipment of the hydropower plant.
- After construction, the ownership of the hydropower plant is transferred to EEA, including all assets and obligations (such as loan repayment). EEA operates the hydropowerplant in accordance with the observed flows in the River Nile and the headpond conditions as defined by the MOPWWR/RGB.

One of the main tasks of the Steering Committee will be to promote the efforts for financing of the project with the Ministries involved.

For the *foreign currency* component of the loans, both Ministries (MOPWWR and MEE) need to apply for funding to the Ministry of International Co-operation, which has the responsibility of organizing and allocating international financing as outlined on Figure 8-5. The loan application must also be approved by the Cabinet of Ministers and the People's Assembly. The Ministry of Finance and the Ministry of International Co-operation then instruct the Central Bank to release the funds.

Figure 8-5: FUND RELEASE PROCEDURE




Regarding the loans foreseen by KfW, there is a financial assistance agreement between the Governments of Egypt and Germany. Responsible for the agreement on the Egyptian side is the Ministry of International Co-operation and on the German side the Ministry for Economic Co-operation.

For financing of the *local currency* component, both the MOPWWR and MEE need to apply to the Ministry of Planning and the Ministry of Finance. Once the project is approved it will appear in the 5 year plan for 1997-2001 with the respective budget allocation.

Additionally, the Steering Committee will have to establish the organizational responsibilities for preparation and construction of the project, including the organizational approach for construction and supervision of construction. In this regard the following should be considered:

(i) It is assumed that the organization of the construction of civil works will be entirely within the responsibility of the MOPWWR/RGB Sector. When installation of the generation and mechanical & electrical equipment for the hydropower plant starts, HPPEA's personnel will be permanently present on the construction site, co-ordinated by the PIU. During construction design, the responsibility of technical co-ordination between civil works and hydropower equipment will be with the supervising Engineer, who will act according to FIDIC Standard Conditions for the design and supervision of construction works (Parts I and II).

(ii) The responsibilities for financing of the project works are split between the MOPWWR/RGB and HPPEA/EEA. If the public road bridge with road connections is constructed, the responsibility of its financing could also be assigned to the MOPWWR. It is understood that the responsibility for financing of the engineering and of environmental mitigation measures of the entire project with temporary compensations would be with the MOPWWR.

(iii) The responsibility for construction of the transmission line would be with the HPPEA, but expropriation for the foundations and the right-of-way for construction should be managed by the Environmental Group under the responsibility of the MOPWWR/RGB.

(iv) Given the stated intention of the Egyptian Environmental Affairs Agency (EEAA) that the implementation of the Environmental Management Plan is the responsibility of the proponent of the project, the MOPWWR/RGB has started to establish an Environmental Group which must be equipped with sufficient resources and institutional capacity as discussed in Section 7.7.

The set-up of the Environmental Group with personal from Egyptian institutions including employment of Egyptian and foreign specialist staff is given in Volume 7.1, Environmental Impact Assessment, for the purpose of budgeting and to obtain financing. The exact composition of the Environmental Group will be subject to further discussions with the PIU and to the decisions taken within the Steering Committee.

# 9. ECONOMIC ANALYSIS

## 9.1 OBJECTIVES AND APPROACH

The objective of the Economic Analysis is to assess the economic viability of the hydropower development on the basis of revised technical parameters and cost estimates for the recommended layout.

*Scope of the Analysis*
The primary purpose of the Naga Hammadi Barrage Development is to ensure long-term continuation of irrigation water supply by construction of a new barrage. In addition, the new barrage can be used for energy generation by adding a hydropower station.

As the necessity of a water retaining structure for irrigation purposes is undisputed, the economic viability of providing the water retaining structure is of no concern within the context of this analysis. The economic evaluation thus is limited to the hydropower component of the project.

*Approach*
The approach to economic evaluation is similar to the one applied in the Conceptual and Interim Studies, ie. the assessment of economic feasibility is based on the comparison of costs and benefits attributable to the hydropower component of the New Naga Hammadi Barrage Development. The benefits of the hydropower component are represented by the avoided costs of alternative thermal generation.

The approach to economic analysis involves the following steps:

- calculation of the project cost in economic prices
- separation of the hydropower cost by calculating the differential for the New Barrage and the New Barrage without hydropower (the Base Case)
- calculation of the environmental effects attributable to the hydropower component
- calculation of hydropower benefits based on the costs of alternative thermal generation
- calculation of the net benefit based on the comparison of hydropower costs (including environmental effects) and benefits.

The discounting technique is used for comparing costs and benefits, which are given as cash flows on an annual basis for each year over the period of construction and the period of operation of the project, by their present values.

The analysis covers the period of construction of the New Naga Hammadi Barrage plus 50 years of operation, from 2006 (the assumed first year of operation) to 2055. A real discount rate of 6% is used for calculation of the present value, with the reference year for discounting being the year 1996.

*Main Parameters*
Basic parameters and assumptions as used in the Conceptual and Interim Studies have been revised and updated in close cooperation with KfW and MOPWWR.

Major revisions of the scenario settings result from changes in the project design: The headpond level of the New Barrage has been fixed at 65.9 m asl. alternative headpond levels are no longer considered. The power plant will be operated in run-of-river mode, peaking will not be permissible.

Other revisions concern the methodological approach: The New Barrage without hydropower (termed "Base Case 2" in the Interim Study) serves as the "Base Case" for the separation of hydropower costs; rehabilitation of the existing Barrage is no longer considered a project alternative for satisfying the purpose of long term continuation of irrigation. The energy benefits of the project are calculated

separately for firm and non-firm energy. The discount rate for present value calculation was revised downwards from 8% to 6%. The base year for discounting was updated to 1996, with cashflows being discounted to the end of the period instead of the beginning.

The Conceptual and Interim Studies assumed 2004 to be the first year of operation. Realistically, construction cannot start before the year 2000, so that the hydropower plant cannot go into operation before 2006. Beginning of 2006 is therefore now assumed as start of operation.

Investment costs have been maintained on September 1995 level, although the Feasibility Study is submitted in August 1997; it should, however, be noted that over the last few years world market prices for electrical and mechanical equipment remained more or less constant in real terms. Technical input data for the analysis, such as energy and capacity of Naga Hammadi Power Station and extent of side effects, were newly established. Fuel costs of the thermal alternative were also revised according to recent information.

The main scenario settings and key parameters are given in Table 9.1.

Table 9.1: KEY PARAMETERS OF THE ECONOMIC ANALYSIS

| Item | Parameter |
|---|---|
| Discount Rate | 6% |
| Price Base<br>Reference Year for Discounting<br>Start of Operation<br>Evaluation Period | 1995<br>1996<br>Beginning of 2006<br>Construction Period plus 50 Years of Operation (1996-2055) |
| Headpond Level<br>Operating Pattern | 65.9 m asl<br>Run-of-River |
| Base Case for Separation of Hydropower Cost | New Barrage without Hydropower |
| Alternative Plant:<br>- Firm Energy and Capacity<br>- Non-Firm Energy | <br>Gas-Fired CC Plant<br>Steam Plant (Mazout-Fired until 2027, Gas-Fired thereafter) |
| Gas Price until 2013<br>Gas Price Factor (Relation to Oil)<br>Real Crude Oil Price in 2006<br>Exchange Rate | LRMC of Gas<br>0.85<br>17.5 US$/bbl<br>1 US$ = 3.4 LE |

## 9.2 HYDROPOWER COSTS

As explained in Section 9.1, the assessment of economic feasibility of the New Barrage is limited to demonstrating the economic feasibility of the hydropower component. For this purpose only the costs and benefits attributable to the hydropower component of the project are relevant. Therefore these costs have to be separated from the total cost of the project. This is done by deducting the cost of a project without hydropower which only satisfies the purpose of continued irrigation (=Base Case) from the one with hydropower:

Hydropower cost = Cost of project with hydropower - Cost of project without hydropower

The project with hydropower is represented by the new barrage with hydropower ("New Barrage"). The purpose of continued irrigation can be satisfied by the new barrage without hydropower; the "New Barrage without Hydropower" thus represents the Base Case. As explained in Chapter 1,

rehabilitation of the existing Barrage cannot be regarded as a feasible alternative to the construction of a New Barrage.

Total costs of the hydropower component comprise costs for the construction and operation of the project as well as environmental effects attributable to the hydropower component. Both cost categories are described in the following sections.

### 9.2.1 Construction and Operation Cost

The construction and operation costs of the project comprise all costs incurred during the lifetime of the project, before commissioning and after commissioning.

The pre-commissioning component of the project cost is represented by the total investment cost including allowances for physical contingencies and engineering. Investment costs are broken down into civil works and hydromechanical, mechanical and electrical equipment. They include costs for a bridge over the New Barrage. As described in more detail in Section 4.10, two alternative options for cross-river traffic were investigated. The economic analysis is based on the Service Bridge Alternative (both for the New Barrage and the Base Case).

As requested by EEA, the investment costs for the New Barrage include costs for a 220 kV transmission line. Since transmission via a 220 kV line does not represent the least cost solution from an economic point of view, the corresponding costs (as well as the incremental energies resulting from reduced losses) cannot be attributed to the Hydropower Component; they are therefore not considered in the economic analysis. In contrast to the financial analyis, the economic analysis is based on the costs (and energies) for a 132 kV transmission line.

Cashflows of the investment costs are established in accordance with the construction schedules of the respective layouts. Construction starts in mid-2000. The construction period is assumed to be 5.5 years for the New Barrage and 5.0 years for the Base Case. The New Barrage thus will start operation in January 2006, whereas the Base Case could be commissioned half a year earlier; operation and maintenance costs of the Base Case are therefore already considered in the second half of year 2005.

The post-commissioning component of the project cost includes the annual operation and maintenance (O&M) costs throughout the economic life of the project, as well as reinvestment costs. According to common practice, the economic life of the civil works of the New Barrage and the Base Case is considered to be 50 years, and the lifetime of electrical and mechanical equipment 30 years. As a consequence, the equipment costs have to be reinvested once during the evaluation period. At the end of the evaluation period the salvage value of the reinvested equipment is considered as negative cost.

The recurrent annual costs for O&M are calculated as a percentage of the initial investment including the allowances for physical contingencies and engineering. The percentages considered are 0.2% of the cost of civil works and 1.5% of the cost of steel structures, electro-mechanical and electrical equipment. These percentages which are based on international experience have been checked against the actual operation and maintenance cost of Aswan I and II dams (300 MW and 270 MW installed capacity) for the fiscal years 1990/91 to 1993/94. For a power plant the size of the High Aswan Dam (2,100 MW) O&M costs are considerably lower in relation to the initial investment costs; however, for Aswan I and II the actual expenses (9.47 million LE and 2.27 million LE excluding depreciation) were found to be consistent with the assumed percentages.

All costs of the project (as well as all benefits) are expressed in monetary terms at their economic values. As the economic analysis should reflect the true costs of the project to the economy, government subsidies, taxes, duties and other factors that inhibit the pricing of labour and materials

and unskilled labour are valued at their shadow prices with conversion factors of 0.82 and 0.57, respectively; a justification of these values is given in Appendix X1.1.

The cost parameters of the hydropower component of Naga Hammadi Barrage Development are summarized in Table 9.2.

Table 9.2: COSTS PARAMETERS OF THE HYDROPOWER COMPONENT

| Item | | New Barrage | Base Case |
|---|---|---|---|
| Construction Period | | 5.5 Years | 5 Years |
| First Year of Operation | | 2006 | 2005 |
| Economic Lifetime: | Civil Works | 50 Years | 50 Years |
| | Mech. & Electr. Equipment | 30 Years | 30 Years |
| Contingencies (Average) | | 12.8% | 14.5% |
| Engineering | | 9.5% | 9.5% |
| Foreign Component | | 70% | 66% |
| Local Component | | 30% | 34% |
| Unskilled Labour (included in Local Component) | | 0.6% | 0.9% |
| Conversion Factors: | Local Material | 0.82 | 0.82 |
| | Unskilled Labour | 0.57 | 0.57 |
| Annual O&M Cost: | Civil Works | 0.2% | 0.2% |
| | Mech. & Electr. Equipment | 1.5% | 1.5% |

Total investment cost before shadow pricing (including allowances for physical contingencies and engineering) of the New Barrage with a headpond level of 65.9 m asl amounts to 314.6 million US$, while the investment cost of the Base Case is estimated at 170.2 million US$. The difference between the investment cost of the New Barrage as considered here (314.6 million US$) and the actual investment cost to be financed by MOPWWR and HPPEA/EEA as considered in the Financial Analysis (322.86 million US$ before duties, see Table 10.4) is explained by the difference between costs for a 132kV and a 220 kV transmission line. As explained above, the incremental costs for the 220 kV line are not attributed to the Hydropower.

Total investment costs in economic terms (after shadow pricing) amount to 297.5 million US$ for the New Barrage and 159.7 million US$ for the Base Case. The investment cost of the New Barrage in economic terms thus is only 5.4 % less than the investment cost in financial terms (314.6 million US$). This rather small effect of shadow pricing is explained by the low share of the local cost component (30% of total cost). With 1.1% of civil works and 0.6% of total construction cost the share of unskilled labour is almost negligible. The Base Case (which does not include any electrical works with a typically high foreign component) has a local cost component of 34%, so that the application of shadow prices to local material results in economic investment costs about 6.1% below the financial cost (170.2 million US$).

It should be noted that the share of the foreign component in the costs of both the New Barrage and the Base Case as presented here is higher than in the cost breakdown shown in the financial analysis, because fuel for construction vehicles and other equipment are treated as foreign currency component in the establishment of the economic investment cost. This is justified by the fact that diesel fuel is purchased locally at a price that is equivalent to the border price. As a consequence conversion into economic cost by shadow pricing is not necessary. In the financial analysis, in contrast, fuel features as a local cost component.

Table 9.3 presents a summary of the investment costs of the New Barrage and of the Base Case in financial and economic terms, and the present value of the project costs over a lifetime of 50 years.

Table 9.3: SUMMARY OF PROJECT COSTS IN $10^6$ US$

| | New Barrage | Base Case | Hydro Component |
|---|---|---|---|
| 1. Investment Cost [1] | | | |
| Civil Works | 146.3 | 112.9 | 33.5 |
| + Hydromech., Mech. &Electr. Equipm. | 108.5 | 22.9 | 85.6 |
| = Total Construction Cost | 254.8 | 135.7 | 119.1 |
| + Contingencies | 32.5 | 19.7 | 12.8 |
| + Engineering | 27.3 | 14.8 | 12.5 |
| = Total Investment Cost (Fin.Terms) | 314.6 | 170.2 | 144.5 |
| 2. Economic Investment Cost [2] | 297.5 | 159.7 | 137.8 |
| as % of Investm.Cost in Fin.Terms | 94.6% | 93.9% | 95.4% |
| 3. Present Value of Project Cost [3] | | | |
| Investment Cost + Reinvestment | 210.8 | 112.3 | 98.5 |
| + O&M Cost | 20.8 | 6.4 | 14.4 |
| = Project Cost (excl. Environm. Effects) | 231.6 | 118.6 | 113.0 |

1) Includes service bridge only, includes 132 kV line only
2) After application of shadow pricing
3) Discount rate 6%, discounting period 1996-2055

### 9.2.2 Environmental Effects Attributable to the Hydropower Component

*Environmental Effects*

The proposed Naga Hammadi Barrage will have environmental impacts both during and after construction of the Project. These will either be *construction-related* or *headpond-level related.*

The former will be short-term, related almost totally to the period immediately prior to or during construction and mainly confined to the immediate environs of the project site. The principal impacts will include relocation of a number of households and other minor infrastructure on Dom Island, some loss of associated assets (both private and communal), and the temporary expropriation of land on Dom Island, the left bank, and flood channel for construction of the New Barrage and associated works. The latter will be restricted to the period of construction with land-by-land compensation proposed at the end of this period. This will also lead to a loss of agricultural production from these areas and the associated livelihoods of those employed.

The implementation of the project with a headpond level of 65.9 m asl will result in a significant rise in the water level in the reach between the New and existing Barrages. As there will also be a rise in headpond level from the present 65.4 m asl in summer and 65.1 m asl during winter, upstream river levels will also increase over a length of some 70 to 80 km upstream (see Chapter 2.4.1). These will both lead to increased groundwater levels in the Project Area necessitating a mitigation programme to minimise the associated impacts. This programme will incorporate an expansion of the existing tile drainage network and drainage outlets, improvements to sanitation systems, and damp proofing of buildings. Positive impacts would, however, result particularly from reduced energy pumping costs and savings in infrastructure costs for new irrigation areas now under construction downstream of Naga Hammadi.

Additional social surveys, training schemes, and various research projects will also be implemented through the establishment of an Environmental Unit within the MOPWWR to oversee the implementation of the environmental mitigation and compensation programme.

Households which are affected by the project will be compensated in cash or kind for the temporary or permanent loss of land, housing and income. These compensations represent financial costs which have to be considered in the financial analysis, while for the economic analysis the losses or gains

attributable to the project have to be valued in economic terms, ie. at their opportunity cost. The economic costs of loss of land, for example, are expressed in terms of the value of lost production over the entire study period.

However, with few exceptions the environmental effects are identical for the New Barrage and the Base Case as they result from the raised headpond level to 65.9 m asl which is common to both. Where the environmental effects also occur as a result of satisfying the irrigation purpose only, that is through the construction of the New Barrage without Hydropower, these also cannot be attributed to the hydropower. Such environmental effects are therefore irrelevant for the purpose of the economic analysis of the hydropower component of the project and their total extent was not considered further in these specific analyses.

The analysis therefore concentrates on the effects which are different for New Barrage and Base Case; this difference is attributed to hydropower. The relevant costs comprise the economic value of land expropriated permanently or temporarily due to the construction and operation of the hydropower component: land reclaimed constitutes negative cost, ie. benefits.

The environmental effects attributed to the hydropower component are summarized in Table 9.4. Details on their calculation are given in Appendix X1.2. Further information on environmental effects in terms of financial costs are given in Volume 7.1, the Environmental Impact Assessment.

Table 9.4: COST OF ENVIRONMENTAL EFFECTS - HYDROPOWER COMPONENT

| Item | Unit | Duration | | Quantity | | | Unit Price LE [1] | Hydro Compon. LE/year |
|---|---|---|---|---|---|---|---|---|
| | | from year | to year | New Barrage | Base Case | Hydro Compon. | | |
| **Land Permanently Lost** | | | | | | | | |
| Left Bank and Flood Channel | feddan | -6 | 50 | 122.8[2] | 94.4 | 28.4 | 2,509 | 71,256 |
| Towers of Transmission Line | feddan | -2 | 50 | 1.6 | | 1.6 | 2,509 | 4,014 |
| **Land Reclaimed (Left Bank/Flood Channel)** | | | | | | | | |
| With Original Top Soil | feddan | 1 | 50 | 122.8[2] | 94.4 | 28.4 | -2,509 | -71,256 |
| With New Top Soil | feddan | 1 | 50 | 116.8[3] | 52.5 | 64.3 | -2,509 | -161,329 |
| **Lost Land Used for Grazing and Seasonal Cropping** | | | | | | | | |
| Cropping Flood Channel | feddan | -6 | 50 | 55 | 32 | 23 | 724 | 16,652 |
| Grazing Flood Channel | feddan | -6 | 50 | 15 | 11 | 4 | 410 | 1,640 |
| **Land Temporarily Occupied** | | | | | | | | |
| Dom Island Site Installation II | feddan | -3 | -1 | 7 | 4.7 | 2.3 | 2,509 | 5,771 |
| **Relocation and Replacement of Material Assets** | | | | | | | | |
| Houses on Dom Island | no. | -6 | -6 | 40 | 24 | 16 | 14,760 | 236,160 |
| Relocation Saiyalet Nag Dawwa Canal & Left Bank Road | km | -6 | -6 | 0.9 | 0.8 | 0.1 | 320,000 | 32,000 |

1) Exchange rate: 3.4 LE/US$
2) Based on land requirements of a 132 kV transmission line; quantities for a 200 kV line are 123.8 feddan
3) Based on land requirements of a 132 kV transmission line; quantities for a 200 kV line are 115.8 feddan

*Project Costs related to Naga Hammadi Mini Hydropower Plant*

About one year before the commissioning of the Naga Hammadi Barrage Hydropower Development, the Naga Hammadi Mini Hydropower Plant (5.2 MW installed capacity) on the left bank will no longer be operated. The New Barrage powerstation will continue to use the discharge formerly passed through the mini hydropower plant, and the larger units at the New Barrage can more efficiently utilise the available head and water than the small units in the mini power plant.

The plant was only recently rehabilitated and would be able to continue power production until the end of its economic lifetime in 2016 (with 1997 as assumed first year of operation). The construction of Naga Hammadi Barrage therefore results in the loss of annual energy production of 38.8 GWh from the mini power plant for about ten years until 2016. HPPEA intends, however, to reduce the

expected loss in energy by relocating the hydropower equipment to another power station in the Delta Zone at Zefta as soon as the mini plant ceases operation; there it would again contribute to energy generation. Costs of relocation, operation mode, output and share of firm and non-firm energy of the new plant under consideration are not known, so that the net cost of the decommissioning of Naga Hammadi Mini to the economy cannot be determined at this stage.

The effects on the Mini Hydropower Plant are the same whether the New Barrage is implemented with or without hydropower. Therefore they cannot be attributed to the hydropower component of the project and consequently are not considered in the economic analysis.

### 9.2.3 Total Hydropower Costs

With environmental effects considered, the present value of the economic costs of the hydropower component amounts to 112.8 million US$. Table 9.5 below shows the breakdown of total costs.

With an annual energy production of 460.8 GWh available at Naga Hammadi Substation, the hydropower plant generates power at dynamic production costs of 2.63 UScents/kWh. Capacity costs amount to 2,170 US$/kW (based on maximum power of 63.5 MW).

Table 9.5: TOTAL COSTS ATTRIBUTABLE TO HYDROPOWER (IN $10^6$ US$)

| | Capital Cost[1)] | Annual Cost | Present Value[2)] |
|---|---|---|---|
| Civil Works (incl. Conting. and Eng.) | 38.3 | | 24.4 |
| + Equipment (incl. Conting.and Eng.) | 99.5 | | 74.1 |
| = Total Investment Cost | 137.8 | | 98.5 |
| + Annual O&M Cost | | 1.6 | 14.4 |
| = Project Cost (excl. Environm. Effects) | 137.8 | | 113.0 |
| + Environmental Costs | | -0.04 | -0.14 |
| = Total Project Cost | 137.8 | | 112.8 |

1) With shadow pricing 2) Discount Rate 6%, Discounting Period 1996-2055

## 9.3 HYDROPOWER BENEFITS

The benefits attributable to the hydropower component of the Naga Hammadi Barrage Development are represented by the electric energy produced by the project. If the hydropower project is implemented, this will avoid the costs for alternative thermal generation required to satisfy the demand for electric energy. Hence, the benefit of a hydropower project is measured by the cost of generating an equivalent amount of power and energy with the least cost thermal means of generation.

When estimating the benefit of the hydropower project, two aspects have to be considered:

- the type of thermal plant that would be installed as an alternative to the hydropower project (thermal alternative) and its location in relation to the load centre, and

- the thermal energy and capacity equivalent to the energy and capacity provided by the hydropower development option at the load centre.

### 9.3.1 Alternative Plant

The appropriate thermal alternative is the type of plant that is able to provide energy and capacity equivalent to the Naga Hammadi Hydropower Project at least cost to the system. Energy and capacity have to be provided at Naga Hammadi substation which has been defined as the relevant load center (see Appendix N).

Within the Interim Study extensive studies were performed to define the type and location of the appropriate thermal alternative. The alternatives tested included: combined cycle power plant, fired with natural gas; steam power plant, dual-fired with Mazout and natural gas; gas turbine power plant, fuelled with natural gas. The studies resulted in the identification of an addition to a gas-fired 300 MW combined cycle (CC) plant with a high thermal efficiency of 48%, located near Cairo, as the appropriate thermal alternative. This selection is justified by the EEA expansion plan, a least-cost analysis, Egypt's energy policy (which promotes the use of gas in thermal power stations in order to save Mazout and Sollar) and the extension of the gas pipeline grid. The gas-fuelled thermal alternative to Naga Hammadi project would have to be installed near existing gas power plants in the Cairo region where gas pipelines are available. Calculations confirmed that the generation costs including transmission losses incurred between Cairo and the Naga Hammadi substation are still lower than the generation costs of a Sollar-fired CC plant in Assiut

Specific investment costs of the combined cycle plant have been revised downwards from 750 US$/kW (the parameter used in the Interim Study) to 650 US$/kW, since world market prices for CC plants decreased considerably during recent years.

The energy benefits are calculated separately for firm and non-firm energy. While the CC plant identified as thermal alternative for firm capacity also represents the appropriate alternative for supplying firm energy, for non-firm energy it is assumed that it could be replaced by the least-efficient steam plant in EEA's system, which is represented by a 300 MW Mazout-fired steam plant with an efficiency of 33% (average of existing old plants). According to EEA's expansion plan, several plants of this type will be retired by the year 2027, and thereafter gas-fired steam units with a slightly higher efficiency of 36% will be the least-efficient plant.

Fuel costs for the alternative type of plant are discussed in detail in Appendix X3. The key parameters for the thermal alternative are summarized in Table 9.6.

Table 9.6: PARAMETERS FOR THE THERMAL ALTERNATIVE

| Item | Unit | Firm Capacity and Energy | | Non-Firm | |
|---|---|---|---|---|---|
| | | | | until 2027 | >2027 |
| Type of Plant | - | Combined Cycle | | Steam | Steam |
| Fuel | - | natural gas | | Mazout | nat.gas |
| Typical Unit Size | MW | 300 | | 300 | 300 |
| Availability | % | 86 | | 83 | 83 |
| Thermal Efficiency | % | 48 | | 33 | 36 |
| Capital Costs | US$/kW | 650 | | N.A. | N.A. |
| Fixed O&M Costs | % of inv. | 1.0 | | N.A. | N.A. |
| Variable O&M Costs | US$/MWh | 2.0 | | 2.4 | 2.4 |
| Heat Rate (ISO) | kJ/kWh | 7,500 | | 10,909 | 10,000 |
| Station Use | % | 2.6 | | 5 | 5 |
| Derating due to High Temp. | % | 2.5 | | N.A. | N.A. |
| | | until 2013 (LRMC) | thereafter | | |
| Fuel Price | US$/MBTU | 1.51 | 2.64 | 2.09 | 2.64 |
| Fuel Cost | US$/MWh | 10.76 | 18.76 | 21.59 | 25.02 |

### 9.3.2 Gas Price

The economic price of gas can be determined by two alternative approaches:

(1) According to the LRMC (Long Run Marginal Cost) Approach, gas is priced on the basis of the cost of gas production and transmission.

(2) According to the Avoided Cost Approach, the price of gas is equivalent to the price of alternative fuels that are displaced by gas.

Which approach is appropriate depends on the supply of gas. When the supply of gas is sufficient to meet demand at a cost below the cost of alternative fuels, then the economic cost of gas is given by the LRMC of production and transmission. When the supply of gas is not sufficient to meet demand at a price lower than that for alternative fuels, then the economic price of gas is given by the value of alternative fuels displaced by gas.

Recent studies by British Gas have shown that the LRMC approach could be applied in the short range from now to 2013. The correct determination of the LRMC of gas was the subject of several studies, and is still discussed controversially. Therefore it is considered that the alternative approach of linking the gas price to the price of oil (which had been applied in the Interim Study) should be given at least equal regard as the LRMC approach. This was then applied for the period after 2013, the study horizon of the LRMC studies.

### 9.3.3 Equivalent Capacity and Energy

The hydropower project and the thermal alternative must provide equivalent services in order to be comparable, ie. the thermal plant must provide the same capacity and the same energy at similar quality of supply as the hydropower plant. This must be the case for the load center at which the capacity and energy are required which in this case is Naga Hammadi Substation.

When determining the capacity of the thermal alternative it has to be considered that the installed capacity of the hydropower plant is not available on a constant basis. It is therefore common practice to use the dependable capacity of a hydropower project for the calculation of the capacity costs of the thermal alternative. The dependable capacity here is defined as firm if the power can be made available to the system on a daily basis with 90% availability throughout the year.

Since the thermal alternative to Naga Hammadi Power Plant is assumed to be located in Cairo, the energy losses resulting from transmission between Cairo and Naga Hammadi have to be added to the energy and capacity available in Naga Hammadi, in order to make both hydro power plant and thermal alternative comparable. For details of the calculation see Appendix X2.

In Table 9.7 energy and capacity of the New Barrage are presented for Naga Hammadi substation, the point at which the energy and capacity generated at the New Barrage would be submitted to the system, as well as for Cairo, the location where the energy and capacity of the alternative plant would be submitted to the system.

Since only the costs of a 132 kV transmission line are attributed to the Hydropower Component of the project, energy calculations consider transmission losses between power plant and substation from a 132 kV line which are higher than those of a 220 kV line (ie. 0.63% of average energy as compared to 0.23%). Energies and capacities in Cairo include the transmission losses on the 500 kV system by which equivalent energy can be submitted at Naga Hammadi Substation.

Table 9.7: ANNUAL ENERGY GENERATION AND DEPENDABLE CAPACITY AT NH SUBSTATION AND THERMAL ENERGY AND CAPACITY TO BE PROVIDED IN CAIRO

| Item | Unit | Firm | Non-Firm | Total |
|---|---|---|---|---|
| **At NHB Site:** | | | | |
| Maximum Power | MW | | | 63.52 |
| Dependable Capacity | MW | 41.76 | | |
| Average Energy | GWh/yr | 329.27 | 136.72 | 465.99 |
| Plant Factor | - | 0.837 | 0.837 | 0.837 |
| Station Use | GWh/yr | 1.65 | 0.68 | 2.33 |
| Transmission Losses (0.62%)[1] | GWh/yr | 2.04 | 0.85 | 2.89 |
| **At Naga Hammadi Substation:** | | | | |
| Average Energy | GWh/yr | 325.58 | 135.19 | 460.77 |
| Present Value of Energy[2] | GWh | 3,037 | 1,261 | 4,299 |
| **Energies and Capacities in Cairo**: | | | | |
| Additional Transmission Losses | GWh/yr | 8.0 | 3.3 | 11.3 |
| Average Energy to be Delivered | GWh/yr | 333.6 | 138.5 | 472.1 |
| Present Value of Energy[1] | GWh | 3,122 | 1,292 | 4,404 |
| Power to be Delivered in Cairo | MW | 42.30 | | 42.30 |

1) Transmission losses calculated for 132 kV line - for values applying to 220 kV line see Chapter 10, Table 10.5.
2) Discount Rate 6%, Discounting Period 1996-2055

### 9.3.4 Avoided $CO_2$ Emissions

Hydroelectric power generation prevents carbon dioxide ($CO_2$) emissions which would otherwise result from the use of thermal plants, since present thermal plant technology does not include the recovery of $CO_2$ from the flue gases. Negative impacts of $CO_2$ emissions include effects such as global warming.

The commitments to reduce $CO_2$ emissions made at the 1992 Rio Conference suggest that power utilities and international financing agencies should include the external costs associated with $CO_2$ emissions into the economic analyses of power projects. Due to methodological problems, the economic costs of $CO_2$ emissions cannot yet be determined exactly; cost estimates of $CO_2$ abatement measures (such as reforestation etc.) range between 3 US$ and 20 US$ per ton of $CO_2$ released by thermal generation. Therefore no attempt is made here to monetarize $CO_2$ related costs of the thermal alternative and to include them in the net benefit calculation. The effects are described only in quantitative terms.

$CO_2$ emissions by a gas fired power plant amount to 56 tons per $10^{12}$J, resulting in 0.42 tons per MWh. A Mazout-fired steam power plant has even higher emissions of $CO_2$: about 78 tons per $10^{12}$J (0.85 tons per MWh). For the thermal alternative of Naga Hammadi with annual gross generation of 497 GWh (351 GWh from gas-fired CC plant, 146 GWh from Mazout-fired, after 2027 gas-fired, steam plants), this results in an emission of some 272,000 tons of $CO_2$ annually, as shown in the table below. The present value of avoided $CO_2$ emission over the lifetime of the project amounts to 2,440,000 tons (calculated by multiplying the $CO_2$ output in tons per GWh with the present value of thermal generation in GWh).

Table 9.8: AVOIDED $CO_2$ EMISSIONS

| Alternative Energy<br>Period<br>Alternative Plant<br>Fuel | | Firm<br>2006-2055<br>CC<br>Natural gas | Non-Firm<br>2006-2027<br>Steam<br>Mazout | Non-Firm<br>2028-2055<br>Steam<br>Natural gas | Total<br>2006-2055 |
|---|---|---|---|---|---|
| $CO_2$ Output of Fuel | kg/GJ | 56 | 78 | 56 | |
| Heat Rate of Plant | kJ/kWh | 7,500 | 10,909 | 10,000 | |
| $CO_2$ Output of Plant | ton/GWh | 420 | 851 | 560 | |
| Annual Energy to be Provided in Cairo | GWh/ year | 333.6 | 138.5 | 138.5 | 472.1 |
| Annual Gross Energy of Th. Altern. in Cairo | GWh/ year | 351.5 | 145.8 | 145.8 | 497.3 |
| Present Value of Gross Energy in Cairo | GWh | 3,279 | 1,039 | 321 | 4,639 |
| Annual Emission of Thermal Alternative | $10^3$ tons/ year | 148 | 124 | 82 | 272 (<2027)<br>229 (>2027) |
| Present Value of $CO_2$ Output of Thermal Alternative | $10^6$ tons | 1.38 | 0.88 | 0.18 | 2.44 |

### 9.3.5 Total Hydropower Benefits

The resulting benefits of the hydropower project as represented by the capacity and energy cost of alternative thermal generation are summarized in Table 9.9 below.

Table 9.9: HYDROPOWER BENEFITS[1)]

| Item | Unit | Parameter |
|---|---|---|
| Capacity Benefits | PV $10^6$ US$ | 29.6 |
| Energy Benefits | PV $10^6$ US$ | 91.5 |
| Total Benefits | PV $10^6$ US$ | 121.1 |
| Energy at NH Substation | PV GWh | 4,299 |
| Dynamic Production Cost of Thermal Alternative | UScents/kWh | 2.82 |
| Avoided $CO_2$ Emissions | PV $10^6$ tons | 2.44 |

1) Discount Rate 6%, Discounting Period 1996-2055

## 9.4 ECONOMIC ANALYSIS

### 9.4.1 Economic Indicators

The economic analysis is based on the following set of assumptions (main case), as discussed and agreed with KfW and EEA:

- discount rate 6%, discounting period 1996-2055, base year for discounting 1996
- gas-fired CC plant as thermal alternative for firm energy and capacity, steam plant as alternative for non-firm energy (Mazout-fired until 2027, gas-fired thereafter)
- gas price equivalent to LRMC of gas until 2013, thereafter gas price per MBTU linked to the crude oil price per MBTU at a factor of 0.85 (on the basis of a crude oil price of 17.5 US$ per barrel)
- Mazout price per barrel linked to crude oil price at a factor of 0.7.

The relevant economic indicator for the assessment of economic viability is the net benefit of the hydro component of the project. As suggested by KfW, other indicators such as Benefit/Cost Ratio and Economic Internal Rate of Return are no longer considered.

The net benefit (or Net Present Value, NPV) is given by the difference between the present value of hydro costs (= cost differential between New Barrage and Base Case) and the present value of hydro benefits (= cost of thermal alternative). The project is economically feasible if its net benefit at a given discount rate is positive.

The dynamic unit cost (or Dynamic Production Cost, DPC) also represent a valuable indicator of economic feasibility when compared to the DPC of alternative generation. The project is feasible if the DPC of the hydro component is lower than the DPC of thermal generation.

Benefits of avoided $CO_2$ emissions are not factored into the net benefit calculation but treated as a separate benefit.

Table 9.10 summarizes the results of the economic analysis.

Table 9.10: SUMMARY OF RESULTS FOR THE MAIN CASE[1)]

| Item | Unit | Parameter |
|---|---|---|
| Hydro Cost for Construction and Operation | PV $10^6$ US$ | 113.0 |
| Environmental Cost | PV $10^6$ US$ | -0.14 |
| Total Hydro Cost | PV $10^6$ US$ | 112.9 |
| Capacity Benefit | PV $10^6$ US$ | 29.6 |
| Energy Benefit | PV $10^6$ US$ | 91.5 |
| Total Benefit (Cost of Thermal Alternative) | PV $10^6$ US$ | 121.1 |
| Net Benefit | PV $10^6$ US$ | 8.2 |
| Annual Energy | PV GWh | 4,299 |
| Dynamic Production Cost Hydro | UScents/kWh | 2.63 |
| Dynamic Production Cost Thermal | UScents/kWh | 2.82 |
| Avoided $CO_2$ Emissions | PV $10^6$ tons | 2.44 |

1) Discount Rate 6%, Discounting Period 1996-2055

### 9.4.2 Sensitivity Analysis and Critical Value Analysis

In order to assess the effect of changes in the basic parameters and assumptions on the economic indicators, a sensitivity analysis was undertaken. The variations of parameters and assumptions include:

- Variation in the discount rate; alternative discount rates of 4% and 8% are applied, the different rates reflecting different opportunity costs of capital. Furthermore the equalizing discount rate is identified; at this rate the present value of costs equals the present value of benefits so that the net present value is zero.

- Changes in investment costs; increases and reductions in construction cost could arise in the intervening period prior to contract signature; the changes in costs depend on changes in quantities in later design stages and on the market situation for heavy construction. In the sensitivity analysis changes of ±10% are tested. Furthermore the critical value is determined, ie. the maximum investment cost increase at which the net benefit is still positive.

  In addition the effect of investment costs for a 220 kV line instead of a 132 kV line is tested. The incremental costs for a 220 kV line correspond to a 5.6% cost increase of the Hydro Component; the increase in voltage from 132 kV to 220 kV also results in increased average energy at Naga Hammadi substation due to reduced losses between power plant and substation and thus to marginally higher benefits from avoided thermal generation.

- Changes in generation: increases and reductions in generation could result from changes in operation conditions. In the sensitivity analysis changes of ±10% are tested. The critical change in generation is tested as well.

- Different assumptions concerning the development of thermal fuel prices; except for the gas price until 2013 which is based on the LRMC of gas, fuel prices for thermal generation are linked to the crude oil world market price. Different assumptions on the development of the crude oil price take account of the uncertainty of any long term price forecast. In the sensitivity analysis changes of ±10% of the oil price are tested (15.75 US$ and 19.25 US$ instead of 17.5 US$ per barrel). Again, the critical value for this parameter is identified.

The results of the sensitivity analysis and the critical value analysis are presented in Tables 9.11 and 9.12 and in Figures 9-1 to 9-4.

When the net benefit of the project is positive, the avoided $CO_2$ emissions represent an additional benefit to the monetary benefits expressed by the net present value. For those cases with a negative net benefit (=net loss), the $CO_2$ benefit can be set in relation to the net loss incurred by the project, in order to calculate the costs of avoiding $CO_2$ emissions: At a discount rate of 8%, the costs of Naga Hammadi exceed the costs of the thermal alternative (the net benefit is negative); however, by investing into Naga Hammadi instead of a thermal alternative, a considerable amount of $CO_2$ could be avoided at a cost of 6.17 US$/ton (calculated as present value of excess costs in relation to present value of avoided $CO_2$ emissions in tons). The decision-makers thus can choose between the options of building Naga Hammadi at an extra cost of 6.17 US$ per avoided ton of $CO_2$ or invest into an apparently cheaper combined cycle plant and accept its negative environmental impact. The results of the respective calculations are shown in Table 9.13.

Table 9.11: SUMMARY OF RESULTS OF SENSITIVITY ANALYSIS

| Discount rate | 4% | **6%** | 8% |
|---|---|---|---|
| | NPV (in $10^6$ US$) | | |
| Main Case | 47.43 | **8.23** | -9.99 |
| Cost Variation +10% | 32.92 | **-3.07** | -19.13 |
| Cost Variation -10% | 61.95 | **19.53** | -0.85 |
| Cost of 220 kV Line Considered | 39.59 | **2.20** | -14.81 |
| Generation Variation +10% | 62.64 | **17.38** | -4.14 |
| Generation Variation -10% | 32.22 | **-0.92** | -15.84 |
| Fuel Price Variation + 10% | 59.27 | **15.00** | -5.87 |
| Fuel Price Variation - 10% | 35.60 | **1.45** | -14.11 |
| | DPC in UScents/kWh | | |
| Main Case Hydro | 2.08 | **2.63** | 3.24 |
| Main Case Thermal | 2.76 | **2.82** | 2.89 |

Table 9.12: RESULTS OF CRITICAL VALUE ANALYSIS

| Item | Critical Value[1)] |
|---|---|
| Equalizing Discount Rate (Main Case) | 6.72% |
| Equalizing Discount Rate (220 kV line considered) | 6.19% |
| Critical Increase in Cost | 7.28% |
| Maximum Investment Cost | 138 x $10^6$ US$ |
| Critical Change in Generation | -9.0% |
| Decrease in Price of Fuel of Thermal Alternative | -12.4% |
| Maximum Crude Oil Price | 15.4 US$/bbl |

1) For discount rate 6% (except for equalizing discount rate)

Table 9.13: COSTS OF AVOIDED $CO_2$ EMISSIONS FOR CASES WITH NEGATIVE HYDRO NET BENEFIT

| Sensitivity Case | NPV of Hydro Project $10^6$ US$ | PV of Avoided $CO_2$ $10^6$ ton $CO_2$ | Cost of Avoiding $CO_2$ US$/ton $CO_2$ |
|---|---|---|---|
| Discount rate 8% | -9.99 | 1.62 | 6.17 |
| Investment cost +10% | -3.07 | 2.44 | 1.26 |
| Generation -10% | -0.92 | 2.20 | 0.42 |

Figure 9-1: NET BENEFIT FOR ALTERNATIVE DISCOUNT RATES

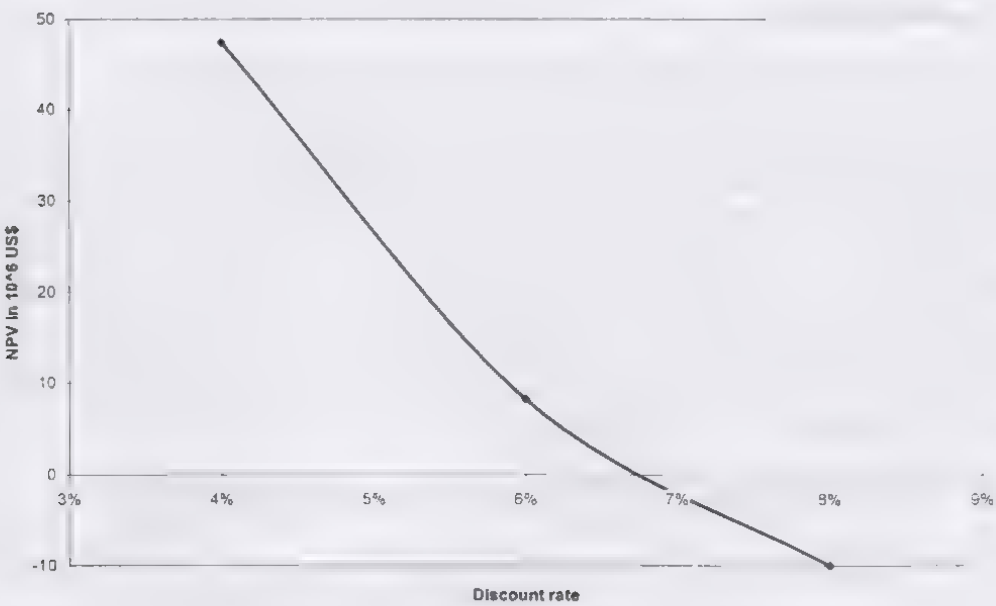



Figure 9-2: NET BENEFIT AT ALTERNATIVE INVESTMENT COST LEVELS FOR ALTERNATIVE DISCOUNT RATES

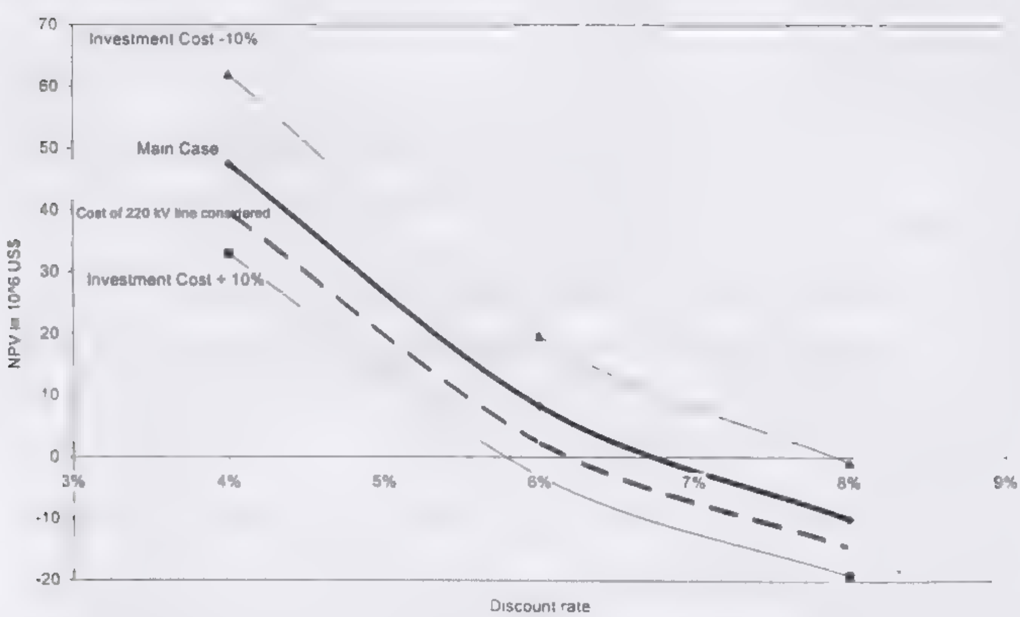


Figure 9-3: NET BENEFIT AT ALTERNATIVE GENERATION LEVELS FOR ALTERNATIVE DISCOUNT RATES

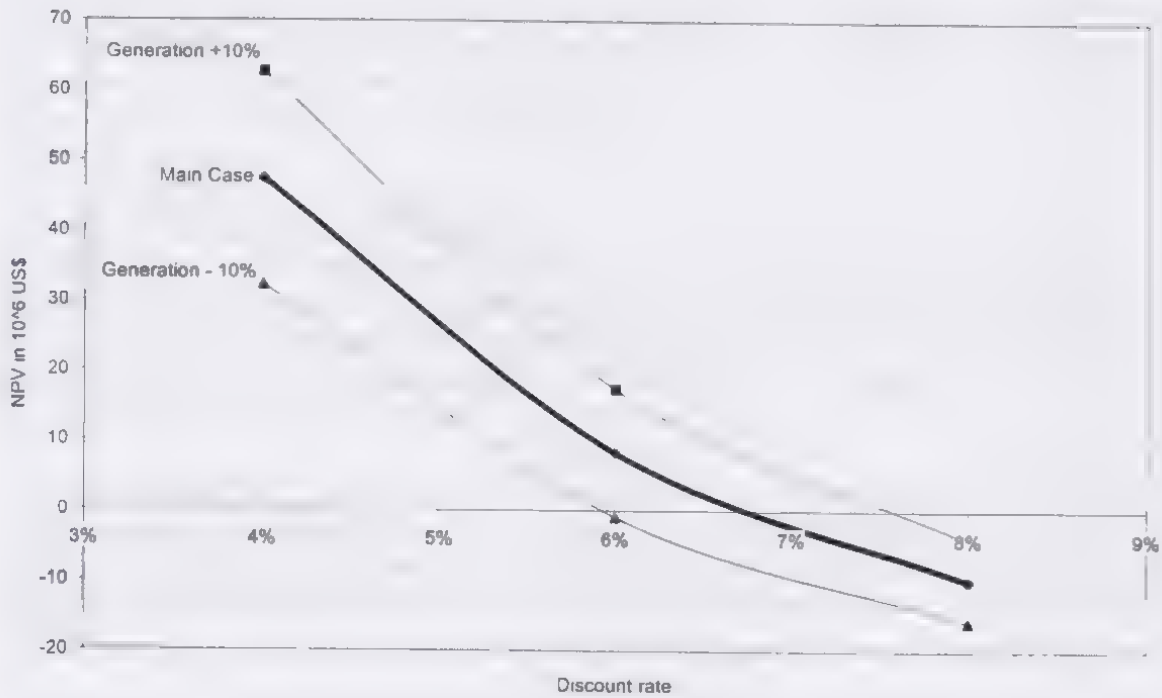



Figure 9-4: NET BENEFIT AT ALTERNATIVE CRUDE OIL PRICES FOR ALTERNATIVE DISCOUNT RATES

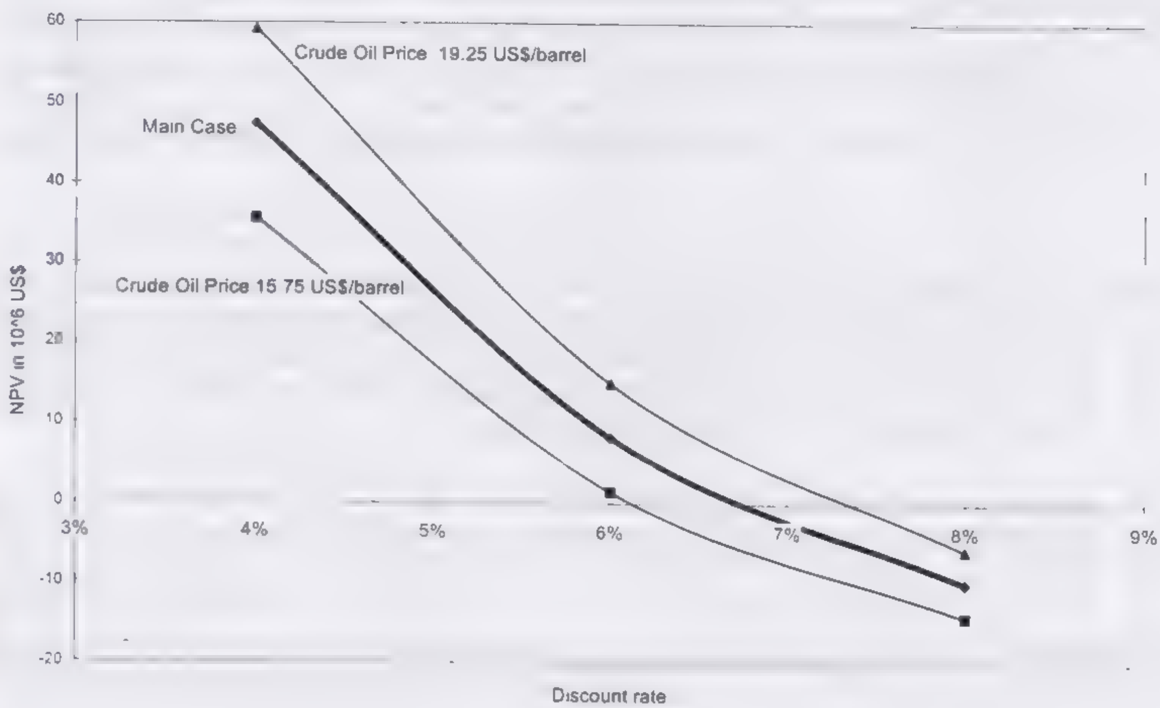
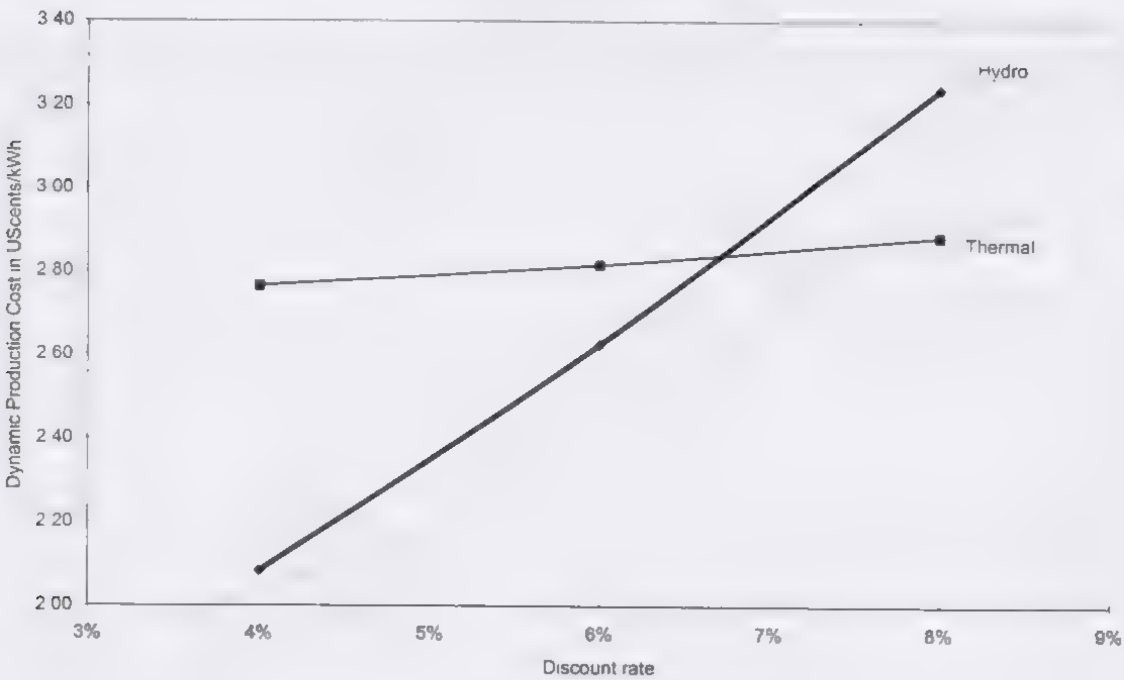


Figure 9-5: DYNAMIC PRODUCTION COST AT ALTERNATIVE DISCOUNT RATES




### 9.4.3 Conclusions

The results of the economic analysis for the Hydropower Component of the New Barrage can be summarized as follows:

- The dynamic production cost of the Hydropower Component is clearly below the dynamic production cost of alternative thermal generation (based on a discount rate of 6%). Thus, the hydro component of the Naga Hammadi Barrage Development is economically feasible.

- The result is rather robust. Even a 10% reduction in the base crude oil price does not render the project unfeasible at a discount rate of 6%. Only at an 8% discount rate does an oil price reduction result in a negative net benefit of the project. If the costs of the 220 kV line were attributed to the Hydropower Component of the New Barrage, the project would still be feasible at a discount rate of 6%. A 10% increase in the investment costs, however, renders the project unfeasible at a 6% discount rate.

- The economic feasibility of the project is strongly influenced by the fuel costs of thermal generation. The analysis is based on the rather conservative assumption that the crude oil price will remain at the 1995 level 17.5 US$/barrel and not increase over time. Considering the growing demand for energy worldwide and the limited supply of thermal energy resources, it may realistically be assumed that the crude oil price will rather increase than decline, so that the feasibility of the hydropower project is ensured in the long run.

- The environmental benefits of $CO_2$ reduction provide an important argument for the implementation of the project. These benefits could even provide a strong incentive for the donor countries to promote the project in view of the commitments made at the Rio Conference in 1992.

# 10. FINANCIAL ANALYSIS

## 10.1 GENERAL

As explained in the foregoing chapters, the New Barrage serves two main purposes, firstly the continued safe supply of irrigation water to large agricultural areas which is the responsibility of the MOPWWR and, secondly, the generation of energy which is under the responsibility of EEA. The legal status of each of the sectors involved in the project determines whether a financial analysis is possible and can be meaningfully carried out.

In the case of the New Barrage, and unlike previous combined irrigation/power projects, each Ministry will finance its own component according to the share agreed between the Ministries. According to the Agreement Concerning Execution of Naga Hammadi Barrage and Hydropower Plant Project of June 26, 1996 between MOPWWR and EEA, the items to be financed by HPPEA/EEA comprise the Generation Equipment, Mechanical Equipment and the Electrical Works, and the remaining costs of the project - about two-thirds of the total cost - will be borne by MOPWWR.

As there are no fees for the use of irrigation water, MOPWWR has no sources of revenue and as a consequence cannot service loans. Consequently, capital expansion and operations in the irrigation sector are financed by the Ministry of Finance, which also services the loans. Financial contributions of the Government for capital expansion of the MOPWWR are considered a non-refundable state budget allocation.

In the past, the Government has been heavily involved in the financing of power projects. These policies are changing and now the Government expects EEA, which has independent sources of revenue, to arrange and service debt for its capital expansion program.

As only EEA, of those sectors involved in the New Barrage project, has its own sources of income through which loans and interest have to be amortized, only the power component of the project is subject to financial analysis.

## 10.2 THE OWNER OF THE HYDROPOWER COMPONENT

### 10.2.1 EEA's Responsibilities and Supply Situation

EEA, which is responsible for generation and transmission of almost all the electricity in Egypt and whose organization is shown in Appendix Y1, distributes electricity to:

- 8 Electricity Distributing Companies (EDCs), reporting to EEA,
- 8 major industrial consumers at 220 kV/132 kV, and
- about 20 industrial and agricultural consumers at 11 kV or 66 kV.

The electricity distributing companies purchase power in bulk, mainly at 66 kV and 33 kV level, from over 750 points of supply.

In 1995/96 electricity sales by EEA were 49,531 GWh, corresponding to revenues of 4,730 million LE. Since 1985, when they amounted to 26,168 GWh, sales have grown at an average rate of over 6% per year, with the highest growth rates taking place before 1989. In the early 90's, the annual growth rates have been about 4%, while in the last two years they again rose to over 6%. For the development of generation and demand see also Appendix N.

With 463 GWh annually, electricity production at Naga Hammadi will represent less than 1% of EEA's total production.

### 10.2.2 EEA's Financial Performance

Financial statements of EEA for the years 1991/92 to 1995/96, which were provided by EEA to KfW, are shown in Appendix Y2. In fiscal year 1995/96, operating revenues were 4,730 million LE, having increased steadily from 2,509 million LE in 1991/92.

The annual operating income, after depreciation but before interest expenses, amounts to 1,653 million LE. Although interest costs have grown to 1,099 million LE per year, net operating income has been positive over past years, reaching 17% of revenues in 1995/96.

These figures do not yet reflect EEA's increased responsibilities for financing of new projects, which could turn the so-far positive situation - as reflected in the financial statements - into negative. As a consequence of such expected and inevitable requirements, the revenues would have to be increased.

The Government of Egypt is discussing the privatisation of the state-owned agencies, in particular in the industrial and banking sectors. The future directions of the Government's policies in the electricity subsector are not well defined at present, but the following assumptions can be safely adopted in the present report:

- Irrespective of the future structure of the subsector, state budget financing of power projects will be drastically reduced.

- EEA will have to rely on its own financing power to fund generation system expansion projects.

- It cannot be excluded that EEA will have to pay income taxes in the future.

From this background, it is important to elaborate a project implementation by which the New Barrage power project does not constitute a financial burden to EEA.

### 10.2.3 EEA's Revenues from Electricity Sales

EEA sells electricity to four major consumer categories:

- Ultra High Voltage (500, 220, amd 132 kV)
- High Voltage (66 and 32 kV)
- Medium Voltage (22 and 11 kV)
- Distribution Companies, at Medium Voltage (22 and 11 kV).

The average revenues per consumer category are lowest for Ultra High Voltage consumers (6.31 pta/kWh in 1995/96) and highest for Medium Voltage consumers (17.37 pta/kWh in 1995/96). The average over all consumer categories was 9.54 pta/kWh in 1994/95 and remained at this level in 1995/96. The development of average revenues is shown in Table 10.1.

Table 10.1: AVERAGE UNIT REVENUES OF EEA BY CONSUMER CATEGORY (in pta/kWh)

| | 1993/94 | 1994/95 | 1995/96 |
|---|---|---|---|
| Ultra High Voltage | 6.34 | 6.31 | 6.31 |
| High Voltage | 11.45 | 11.43 | 11.42 |
| Medium Voltage | 17.36 | 17.34 | 17.37 |
| MV Distribution Companies | 9.69 | 9.96 | 9.96 |
| EEA Average | 9.29 | 9.54 | 9.54 |

The average unit revenue relevant for electricity sales from the New Naga Hammadi Hydropower corresponds to EEA's average unit revenue minus the average cost of transmission between UHV MV supply, the latter amounting to about 2% of the average unit revenue. Deducting this 2% from I average unit revenue of 9.54 pta/kWh results in the relevant tariff of 9.35 pta/kWh to be applied financial analysis of the Project.

In 1990 the Government of Egypt agreed with the international financing community to raise stepwi average electricity tariff (revenue neutral) to the Long Run Marginal Cost of electricity by June I The results of the implementation of this agreement are visible in EEA's profit and loss statement. the average revenue per kWh earned by EEA increased from 3.02 pta/kWh in 1989/90 to 9 54 pta in 1994/95. According to EEA information the average tariff on end-consumer level is 12.7 pta/kWl

The determination of the LRMC has been the subject of several studies and discussions, n concerning the type and cost of the fuel to be used for thermal generation. Under the (rea assumption that in the future natural gas will be the most important fuel for electricity generatio LRMC of electricity will be lower than under the assumption of fuel-oil based generation plants, d the lower cost of gas production. Estimates of the LRMC of electricity range between 9.10 pta (EEA 1995; financing cost and depreciation not included) and 12.97 pta/kWh (RCG/Hagler 1994/95; based on LRMC of gas of 4.11 UScents/m$^3$).

A recent tariff study by EEA and IRG indicates that the following tariffs are required in order to:

| | | |
|---|---|---|
| - | achieve a debt service ratio of 1.5% in 1999 | 12.03 pta/kWh |
| - | achieve a self financing ration of 35% in 1999 | 12.41 pta/kWh |
| - | meet cash requirements in 1999 | 12.19 pta/kWh. |

With a current average revenue of 9.54 pta/kWh EEA covers about 80% of its LRMC

## 10.3 OBJECTIVES OF ANALYSIS AND APPROACH

The objectives of the financial analysis are:

- to determine the financial feasibility of the project from the point of view of EEA, and
- to investigate if with EEA's supply tariffs the project can be financed.

Similar to the economic analysis, the financial analysis covers only the hydropower component project, i.e. the project component implemented by HPPEA and thereafter overtaken by EE operation. The irrigation component, which is the responsibility of the MOPWWR, is not considerec

With regard to its dual objectives, the financial analysis comprises two parts:

(i) a Dynamic Production Cost (DPC) Analysis, and
(ii) a Cashflow Analysis.

(i) Dynamic Production Cost Analysis

The DPC Analysis serves to assess if the implementation of the hydropower component of the proje benefit for EEA. For this purpose the unit cost of energy generation by the New Naga Har powerplant is compared to the unit cost of alternative thermal generation and with EEA's bulk s tariff. If the DPC of the project are lower than the DPC of thermal generation, the implementation project clearly is of advantage to EEA, since EEA can save costs by hydro instead of thermal gene If the DPC of the project are lower than the bulk supply tariff, the revenues from energy sales v

sufficient to recover the production costs, irrespective of the DPC being higher or lower than the DPC of thermal generation.

The dynamic production costs are defined as the present value of the cost of energy generation divided by the present value of the units generated over the entire lifetime of the project. Similar to the economic analysis a cashflow of project costs is set up, comprising investment and operation costs; but in contrast to the economic analysis, costs are expressed in financial terms, i.e.:

- indirect costs to third parties other than HPPEA/EEA (such as compensation for negative environmental effects) are not considered,
- the investment costs of a 220 kV transmission line as actually to be incurred by HPPEA/EEA are considered,
- shadow prices are not applied, fuel costs are considered in local currency,
- taxes, duties and subsidies are included.

As agreed with KfW and HPPEA/EEA, the DPC are calculated for two cost scenarios:

- COST SCENARIO 1: The Ministry of Energy and Electricity - represented by HPPEA/EEA - bears the costs for implementation, operation and maintenance of the power component which comprise the capital costs for Generation, Mechanical and Electrical Equipment, and Transmission Line and the O&M costs pertaining to this equipment.

- COST SCENARIO 2: All costs attributable to the hydropower component are considered. These costs are given by the differential costs (project costs and environmental effects) for the New Barrage (with hydropower) and the New Barrage without Hydropower (Base Case).

For the purpose of the DPC analysis the costs are expressed in constant prices in order to avoid distortions caused by inflationary effects. Results are shown in US$ and in LE.

(ii) Cashflow Analysis

The cashflow analysis is studied for the Naga Hammadi Hydropower Plant as a self-contained case, with cash inflows by EEA's revenues from electricity sales. As long as the project cashflow does not become negative for a longer period of years and relies on financial contribution from EEA in addition to the revenues from electricity sales, the project will not affect EEA's overall financial position. This is seen as a requirement for the financial viability of the project.

The cashflow comprises project costs as cash outflow and revenues from electricity sales as cash inflow. In addition, financing of the project is considered in the cashflow (loans and equity, if applicable, as cash inflow and debt service as cash outflow).

The cashflow should give a correct indication of the financial situation of the project in any given year with respect to the prices of inputs and outputs. Therefore the cashflow is expressed in current prices (including inflation). Since EEA's accounts are set up in LE, the project cashflow is presented in local currency, with expenses incurred in foreign currency being converted to LE at the current exchange rate between US$ and LE.

As agreed with KfW and HPPEA/EEA, the project cashflow is set up only for COST SCENARIO 1: the costs borne by HPPEA/EEA are the capital costs for Generation, Mechanical and Electrical Equipment, Transmission Line and the O&M costs pertaining to this equipment.

Table 10.2 summarizes the approach to the financial analysis.

Table 10.2: APPROACH TO FINANCIAL ANALYSIS

| Methodology | DPC Analysis | Cashflow Analysis |
|---|---|---|
| Objective | Assessment of Financial Profitability of the Project for EEA | Assessment of Project Impact on Financial Position of EEA |
| - COST SCENARIO 1:<br><br>- COST SCENARIO 2: | Costs to be borne by HPPEA/EEA: Generation Equipment, Mechanical & Electrical Equipment, Transmission Line<br><br>Hydropower Component: New Barrage minus Base Case, both including environmental effects | Costs to be borne by HPPEA/EEA: Generation Equipment, Mechanical & Electrical Equipment, Transmission Line<br><br>n a |
| Prices | Constant Terms | Current Terms |
| Currency | US$ and LE | LE |

The key parameters applied in the financial analysis are summarized in Table 10 3.

Table 10.3: KEY PARAMETERS OF THE FINANCIAL ANALYSIS

| Item | Parameter | |
|---|---|---|
| Price Base<br>Start of Construction<br>Start of Operation<br>Operation and Maintenance Cost<br>Taxes and Duties | September 1995<br>mid-2000<br>January 1, 2006<br>1.5% of Investment Cost<br>5% of Imported Equipment | |
| | DPC Analysis | Cashflow Analysis |
| Discount Rate<br>Reference Year for Discounting<br>Evaluation Period | 5%, 8%<br>1996<br>Construction plus 50 Years of Operation (1996-2055) | n.a.<br>n.a.<br>Construction plus 30 Years of Operation (1996-2035) |
| Alternative Thermal Plant:<br>- Firm Energy and Capacity<br>- Non-Firm Energy | <br>Gas-Fired CC Plant<br>Steam Plant (Mazout-Fired until 2027, Gas-Fired thereafter) | <br>n.a.<br>n.a. |
| Exchange Rate LE/US$<br><br>Local Inflation<br>Foreign Inflation | 3.40<br><br>n.a.<br>n.a. | 3.40 until 1999, fluctuation with inflation differential thereafter<br>8% p.a. until 1999, 5% p.a. thereafter<br>2.5% p.a. |

## 10.4 PROJECT INVESTMENT COSTS AND SHARE OF HYDROPOWER

The costs and disbursement schedules for the construction of the New Naga Hammadi Barrage project with Service Bridge are taken from the relevant tables of Appendix T which are summarized in Chapter 8, Table 8.2. Price basis is September 1995.

Since custom duties have to be paid for the cost of imported generation, electrical and mechanical equipment, 5% (payable in local currency) has been added to the cost for imported equipment. This percentage is an average of different custom duty rates applicable to different types of equipment.

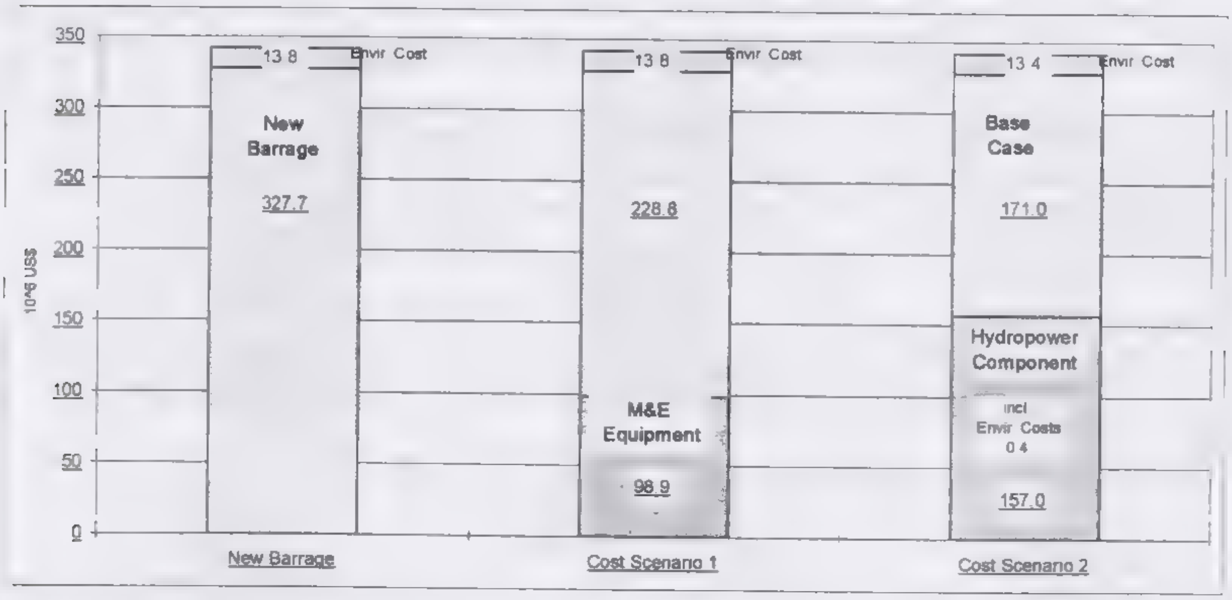
Total costs of the project (including duties) amount to 327.7 million US$ in prices of September 1995 (1,114 million LE). The cost of the New Barrage without Hydropower (Base Case) is estimated at 171.0 million US$ (582 million LE). Deducting the cost of the Base Case from the cost of the New Barrage with Hydropower results in costs of 156.6 million US$ (533 million LE) for the hydropower component, including duties.

Costs for the compensation and mitigation of side effects resulting from the construction works amount to 13.8 million US$; 0.4 million US$ of these costs can be attributed to the hydropower component, thus increasing the hydropower costs to 157.0 million US$ (534 million LE).

The cost share of HPPEA/EEA (generation equipment, mechanical equipment, electrical works and transmission line) represents about 30% of the costs of the New Barrage. These costs total 98.9 million US$ (336 million LE).

The costs of the New Barrage project, the Hydropower Component and the M&E Equipment as agreed to be financed by HPPEA/EEA are compared in Figure 10-1.

Figure 10-1: INVESTMENT COST OF NEW BARRAGE; HYDROPOWER COMPONENT AND M&E EQUIPMENT in constant 1995 $10^6$ US$




The above investment costs are expressed in constant 1995 US$. Until the end of construction, inflation will have considerably increased the total costs of the New Barrage. As agreed with KfW and HPPEA/EEA, foreign prices are assumed to increase by 2.5% annually, and domestic prices by 8% until 1999 and by 5% thereafter. It should be noted that application of these price increase rates to the Naga Hammadi Barrage Project may overstate the price increase to be actually expected, since over the last few years particularly prices for civil works have increased less than the average consumer prices. The price contingencies for the project are thus based on a conservative approach.

Normally, the inflation differential between two countries is reflected in the fluctuation of their currency exchange rate. In the case of Egypt, however, the exchange rate of the LE vis-a-vis the US$ has been constant over the last few years although the local rate of inflation exceeded by far the rate of inflation in the USA. Taking into account this recent development and as agreed with KfW and HPPEA/EEA, it is assumed that the LE/US$ exchange rate will continue to remain constant at 3.40 LE/US$ until 1999; thereafter the LE will be devalued in line with the inflation differential. This leads to the effect that local and foreign cost components in the cashflow increase at different rates although expressed in a single currency - an effect that must be kept in mind when comparing project cost in US$ and LE.

In current terms, i.e. including price increases due to inflation, the costs of the New Barrage (excluding environmental costs) amount to 430.4 million US$ (1,616 million LE), costs of the hydropower component to 206.4 million US$ (785.9 million LE), and costs of the M&E equipment to 128.1 million US$ (487.6 million LE).

Table 10.4 summarizes the project costs as required for the DPC Analysis (Hydro Power Component and M&E Equipment in constant terms) and for the Cashflow Analysis (M&E Equipment in current terms). Interest during construction (IDC) adds to the total financing requirements for the M&E Equipment to be financed by HPPEA/EEA as used in the cashflow analysis. Detailed breakdowns of costs are given in Appendix Y3.

Table 10.4: INVESTMENT COSTS

| | New Barrage | | Base Case | | Hydro Power Component | | M&E Equipment | |
|---|---|---|---|---|---|---|---|---|
| | $10^6$ US$ | $10^6$ LE | $10^6$ US$ | $10^6$ LE | $10^6$ US$ | $10^6$ LE | $10^6$ US$ | $10^6$ LE |
| A Civil Works | 146.3 | 497 5 | 112.8 | 383.7 | 33.5 | 113.8 | 0 0 | 0.0 |
| B Hydrom. Equipment | 28.8 | 97.9 | 21.3 | 72 3 | 7 5 | 25 5 | 0 0 | 0 0 |
| C Mechan. Equipment | 36.6 | 124.3 | 0.0 | 0.0 | 36.6 | 124.3 | 36.6 | 124.3 |
| D Electrical Equipment | 50.0 | 170.0 | 1.6 | 5.4 | 48.4 | 164 5 | 50 0 | 170.0 |
| Physical Contingencies | 33.2 | 112.8 | 19.7 | 66.9 | 13.5 | 45.9 | 8 7 | 29.5 |
| Engineering | 28.0 | 95 2 | 14.8 | 50 2 | 13 2 | 45 1 | 0 0 | 0 0 |
| Subtotal | 322.9 | 1097.7 | 170.2 | 578.5 | 152 7 | 519 2 | 95.2 | 323.8 |
| Duties | 4.8 | 16.4 | 0.9 | 3.0 | 4 0 | 13.5 | 3 7 | 12.6 |
| Construction Cost (constant terms) | 327.7 | 1114.1 | 171.0 | 581.5 | 156.7 | 532.6 | 98.9 | 336.4 |
| *Price Contingencies* [1] | *102.7* | *502.2* | *52.9* | *248.9* | *49 8* | *253.3* | *29 2* | *151.2* |
| *Construction Cost (current terms)* | *430.4* | *1616.4* | *223.9* | *830.4* | *206.4* | *785.9* | *128.1* | *487.6* |
| Environmental Cost | 13.8 | 46.8 | 13.4 | 45.4 | 0.4 | 1.2 | 0.0 | 0.0 |
| *Price Contingencies* [1] | *5.6* | *27.6* | *5.6* | *26.7* | *0.2* | *0.9* | *0.0* | *0.0* |
| *Subtotal Environm. Cost* | *19.6* | *74.4* | *19.0* | *72.1* | *0.6* | *2.1* | *0 0* | *0 0* |
| Total Investment (constant terms) | 341.5 | 1161.0 | 184.4 | 627.1 | 157 0 | 533.8 | 98 9 | 336.4 |
| *Total Investment (current terms)* | *450.0* | *1690.8* | *242.9* | *902.8* | *207.0* | *788.0* | *128.1* | *487 6* |
| *IDC* | *97.1* | *366.0* | [2] | | [2] | | *16.3* | *62.8* |
| *Financing Requirements* | *547.1* | *2056.8* | | | | | *144.4* | *550.5* |

1) for inflation and exchange rate development see text above; price contingencies for Base Case and Hydropower Component given only for comparative reasons (DPC Analysis is carried out in constant terms - without inflation)

2) IDC not calculated

## 10.5 DYNAMIC PRODUCTION COST OF HYDROPOWER

The dynamic production costs are expressed in UScents or piasters per kWh generated by the project. For their calculation, annual streams of costs and energy are set up over the entire lifetime of the project and then discounted to their present values (1996).

The DPC analysis is based on the following:

*General*

- The evaluation covers the years 1996 to 2055, including 50 years of operation after commissioning in 2006. Base year for discounting is 1996.
- According to standard KfW practice, discount rates of 5% an 8% are applied.
- Costs are given in financial terms, ie. including taxes and duties.
- Costs are given in constant terms, ie. excluding price contingencies, with the aim that the resulting DPC is not distorted by inflationary effects.

*For the Hydropower Project*

- Two scenarios for the investment costs to be borne by the Ministry of Energy and Electricity (represented by HPPEA/EEA) are considered:

  COST SCENARIO 1: Generation Equipment, Mechanical and Electrical Equipment, and 220 kV Transmission Line excluding engineering;

  COST SCENARIO 2. all investment and compensation costs attributable to the hydropower component, equivalent to the cost differential between New Barrage and New Barrage without Hydropower (Base Case).

  The respective costs are given under"Total investment cost (constant terms)" in Table 10 4

- Lifetime of the project components is assumed to be 50 years for civil works (only relevant for Cost Scenario 2) and 30 years for equipment. Equipment is reinvested after 30 years, and its salvage value (based on straight line depreciation) is considered as negative cost at the end of the project horizon.

- Operation of the hydropower plant starts in January 2006 after 5.5 years of construction; construction of the New Barrage without Hydropower (only relevant for Cost Scenario 2) is completed within 5.0 years, so that operation starts already in mid-2005.

- The recurrent operation and maintenance costs are calculated as a percentage of the initial investment cost including the allowances for physical contingencies and engineering. The percentages considered are 0.2% of the costs of civil works (only relevant for Cost Scenario 2) and 1.5% of the cost of hydromechanical, mechanical and electrical equipment.

- Annual energy production (measured in Naga Hammadi Substation as the relevant load center) is 462.6 GWh, as shown in Table 10.5.

Table 10.5: ANNUAL ENERGY GENERATION AND DEPENDABLE CAPACITY AT NH SUBSTATION AND THERMAL ENERGY AND CAPACITY TO BE PROVIDED IN CAIRO

| Item | Unit | Firm | Non-Firm | Total |
|---|---|---|---|---|
| **At NHB Site:** | | | | |
| Dependable Capacity | MW | 41.76 | | |
| Average Energy | GWh/yr | 329.27 | 136.72 | 465.99 |
| Station Use | GWh/yr | 1.65 | 0.68 | 2.33 |
| Transmission Losses (0.23%) | GWh/yr | 0 76 | 0.32 | 1 07 |
| **At Naga Hammadi Substation:** | | | | |
| Average Energy | GWh/yr | 326.86 | 135.72 | 462.59 |
| **Energies and Capacities in Cairo**: | | | | |
| Additional Transmission Losses | GWh/yr | 8.0 | 3.3 | 11.3 |
| Average Energy to be Delivered | GWh/yr | 334.9 | 139 0 | 473.9 |
| Power to be Delivered in Cairo | MW | 42.47 | | 42.47 |

*For the Thermal Alternative*

- The appropriate thermal alternative for the Naga Hammadi Hydropower Plant has been defined for the economic analysis (gas-fired combined cycle plant for firm energy and capacity, steam plant for non-firm energy). For detailed justification see Chapter 9 3 1.

- Capacity and energy costs are calculated by the same method as applied in the economic analysis (i.e. taking into account plant availability, station use and transmission losses from Cairo to the relevant load center in Naga Hammadi Substation).

- All technical parameters, cost parameters and fuel consumption of the thermal alternative are similar to those applied in the economic analysis, with the following exceptions:

  - Duty is added on the foreign equipment cost component of the combined cycle plant.

  - Fuel costs of gas and mazout are given in financial terms at the actual purchase price of EEA: 139 LE (41 US$) per 1000 $m^3$ gas, 130 LE (38 US$) per ton Mazout. Presently EEA pays 122.5 LE per 1000 $m^3$ gas, but the gas price is expected to be increased to 139 LE per 1000 $m^3$, which corresponds to the LRMC of gas. These prices correspond to less than 50% of the world market price level for both fuels, which are 85.8 US$ per 1000 $m^3$ gas (assuming that the gas price is linked to the oil price) and 81.3 US$ per ton Mazout in 1995 prices; for discussion see Appendix X3.

Details of the DPC Analysis are presented in Appendix Y4, and Table 10.6 summarizes the results.

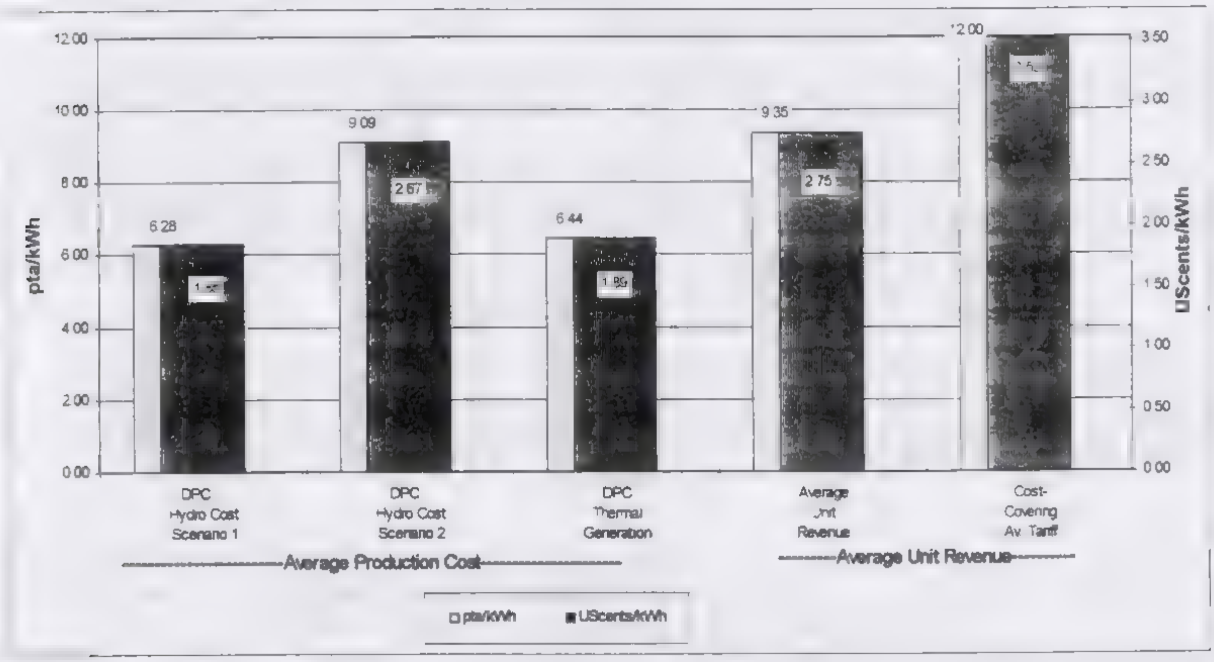
Table 10.6: DYNAMIC PRODUCTION COST OF HYDROPOWER AND OF THE THERMAL ALTERNATIVE

| | Unit | Naga Hammadi Hydropower Project | | | | Thermal Alternative | |
|---|---|---|---|---|---|---|---|
| | | Cost Scenario 1: M&E Equipment | | Cost Scenario 2: Total Hydro Comp. | | | |
| Firm Capacity at NH | MW | 41.76 | | 41.76 | | | |
| Annual Energy at Naga Hammadi Substation | GWh | 462.6 | | 462.6 | | | |
| Investm. Cost (excl. envir.) | $10^6$ US$ | 98.9 | | 156.7 | | 34.2 | |
| Specific Investment Cost | US$/kW | 2,369 | | 3,752 | | 650 [1] | |
| Annual O&M Cost | $10^6$ US$ | 1.56 | | 1.81 | | 1.40 | |
| Annual Fuel Cost | $10^6$ US$ | - | | - | | 4.49 | |
| | Discount Rate | 5% | 8% | 5% | 8% | 5% | 8% |
| Present Value of Costs | $10^6$ US$ | 100.5 | 69.7 | 145.6 | 104.3 | 103.1 | 59.0 |
| *thereof: Environm. Costs* | | | | *0.3* | *0.2* | | |
| Present Value of Energy | $10^6$ US$ | 5.444 | 2.831 | 5.444 | 2.831 | 5.444 | 2.831 |
| Dynamic Production Cost | UScents/kWh | 1.85 | 2.46 | 2.67 | 3.68 | 1.89 | 2.08 |
| | Pta/kWh | 6.28 | 8.37 | 9.09 | 12.53 | 6.44 | 7.09 |

[1] Based on net capacity, before adding duties

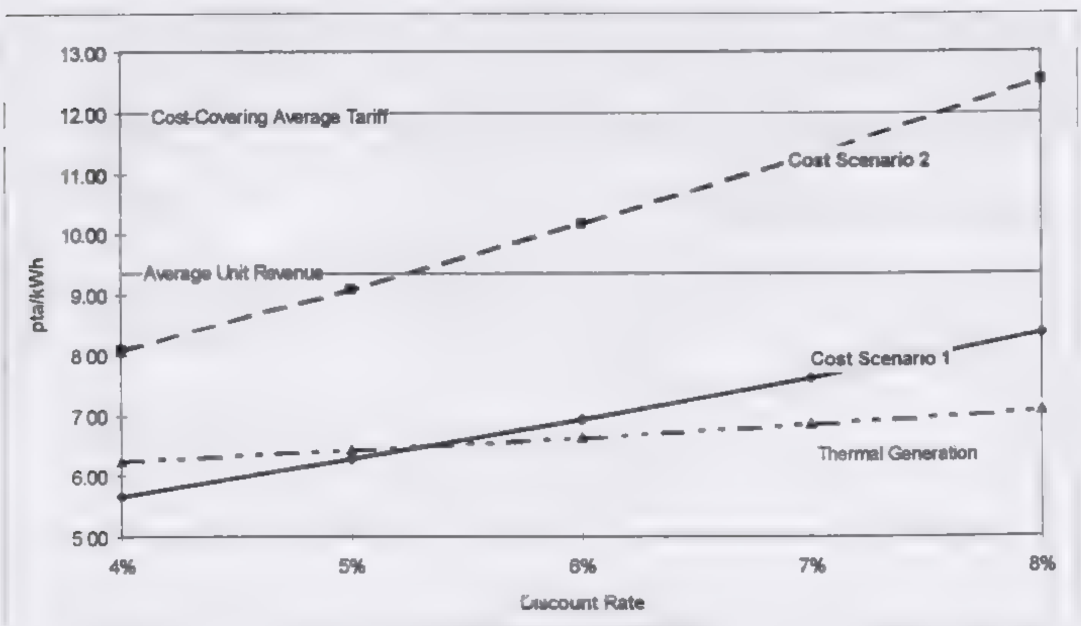
Figure 10-2 compares the DPC of hydro and thermal generation at a discount rate of 5% to alternative tariff levels.

Figure 10-2: DYNAMIC PRODUCTION COSTS AT A DISCOUNT RATE OF 5% AND AVERAGE UNIT REVENUES in pta/kWh and UScents/kWh




In Figure 10-3 the DPC of hydro and thermal generation are shown for alternative discount rates; average unit revenues are also given for comparison.

Figure 10-3: DYNAMIC PRODUCTION COSTS AT ALTERNATIVE DISCOUNT RATES in pta/kWh




The results of the DPC analysis lead to the following conclusions:

- The DPC are distinctly lower for Cost Scenario 1 (equipment only) than for Cost Scenario 2 (Total Hydropower Component). The financial burden for EEA will, therefore, be distinctly lower than when sharing the costs between the irrigation and the hydropower components.

- The DPC of the project (6.28 pta/kWh for Cost Scenario 1 at a discount rate of 5%, 8.37 pta/kWh at 8%) compares favourably with EEA's average unit revenue at the relevant voltage level (9.35 pta/kWh or 2.75 UScents/kWh - see Chapter 10.2.3 above). This implies that EEA will be able to recover the production costs from the sales revenues at present tariffs. If tariffs are increased to LRMC level (12 pta/kWh or 3.5 UScents/kWh), average revenues will exceed dynamic production costs by a factor of 1.9, thus improving EEA's financial position considerably.

- The DPC of hydro generation by Naga Hammadi Hydropower Plant and transmission to the relevant load center at Naga Hammadi Substation (Cost Scenario 1) are slightly lower than the DPC of alternative thermal generation in Cairo and transmission to NH Substation at a discount rate of 5%. For higher discount rates DPC of hydro generation are above those of thermal generation. This is due to the low fuel prices which put hydro generation at an undue disadvantage.

In a sensitivity analysis the effects of changes in key parameters on the DPC of hydro and thermal generation were tested.

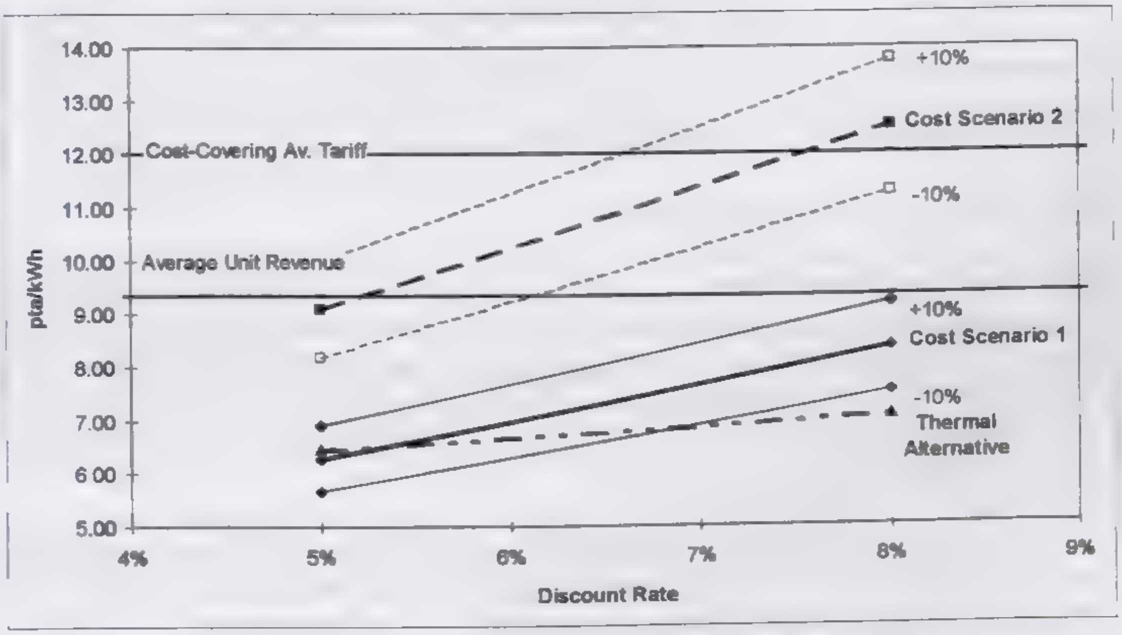
*Changes in Investment Cost*
As shown in Figure 10-4 below, a 10% increase in investment cost has the following effects:

- Cost Scenario 1: At a discount rate of 5%, the DPC of hydro generation is still below EEA's average unit revenue; even at a discount rate of 8% the DPC is still below the average unit revenue, which implies that EEA will be able to recover the production costs from the sales revenues at present tariff. Also, at a discount rate of 8%, the DPC of hydro generation is below the cost-covering tariff level.

- Cost Scenario 2: At a discount rate of 5%, the DPC of hydro generation rises above EEA's average unit revenue, and at a discount rate of 8% the DPC exceeds even the cost-covering average tariff.

If the investment costs are 10% below the estimate, the DPC of hydro generation (Cost Scenario 1) are not only lower than the present average unit revenue, but also below the DPC of thermal generation.

Figure 10-4: DYNAMIC PRODUCTION COSTS FOR ALTERNATIVE INVESTMENT COST LEVELS in pta/kWh



*Changes in Fuel Costs*

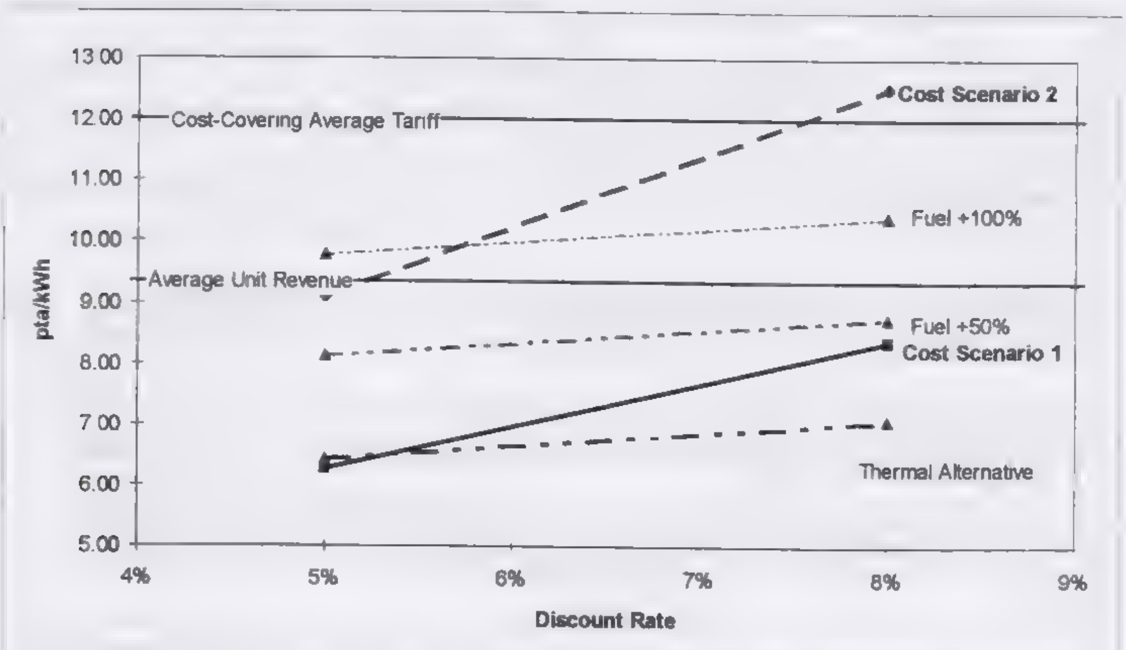
The DPC of thermal generation is mainly determined by the prices of natural gas and Mazout, which - as stated above - are more than 50% below world market level. Figure 10-5 shows the following:

- A 50% increase in fuel prices of thermal generation (bringing the fuel prices to about 75% of world market level) increases the DPC of thermal generation at discount rates of 5% and 8% above the DPC of hydro generation for Cost Scenario 1.

- If fuel is sold to EEA at world market price level, the DPC of thermal generation are considerably higher than the DPC of hydropower generation (Cost Scenario 1) at discount rates of 5% and of 8%.

- The DPC of thermal generation even exceed the DPC of hydro generation (Cost Scenario 2) at a discount rate of 5%, if fuel prices are increased by 100%.

Figure 10-5: DYNAMIC PRODUCTION COSTS OF ALTERNATIVE THERMAL GENERATION FOR ALTERNATIVE FUEL COST ASSUMPTIONS (in pta/kWh)




## 10.6 CASHFLOW ANALYSIS OF HYDROPOWER

### 10.6.1 Sources of Funds And Financing Conditions

Total costs of the New Barrage amount to 430.38 million US$ in current terms (including price contingencies, but excluding environmental costs); including environmental cost, costs of the New Barrage amount to 450.75 million US$. According to the agreement between MOPWWR and EEA, HPPEA/EEA will only be responsible for financing the equipment cost (128.1 million US$ including price contingencies). Therefore, as agreed with KfW and HPPEA/EEA, only this Cost Scenario 1 is considered in the Cashflow Analysis.

No agreement has been reached on the financing of the entire project, but some suggestions are summarized in Appendix Y3. However, it is assumed to be realistic to base the financing plan of EEA's share for this feasibility study on the following assumptions:

*Equity*
Neither EEA nor the Government will provide equity for EEA's share of the project.

*Grants*
Grants from donors such as KfW, World Bank and EIB are not available for the project, as it is not a social sector project, and grant funds and concessional loans (IDA funds) for Egypt are largely exhausted.

*Loans*
The above implies that HPPEA/EEA's share will have to be completely loan financed. The hydropower generation equipment will be financed by the Kreditanstalt für Wiederaufbau (KfW) through the Ministry of Electricity and Energy/HPPEA.

*Foreign Currency Component*
The entire foreign currency component of the equipment will be financed by KfW.

*Local Currency Component*
International lenders usually require a substantial local financing of the local cost component as an indication of the local commitment to the project and of its sustainability. HPPEA/EEA is prepared to finance the entire local component. However, in this particular case the local component may also be financed by KfW. Therefore, two alternative financing options are considered in the cashflow analysis.

*Customs charges*
Foreign donors generally do not finance customs charges; therefore these charges will be financed locally via Egyptian banks.

*Lending Procedures and Financing Conditions*
*a. KfW*
KfW will give a loan to the Central Bank which will on-lend the funds in the currency required to HPPEA. The loan will be repaid by EEA in DM (with the exchange risk borne by EEA). KfW may also provide the funds for the local currency component of the M&E equipment. In this case the Central Bank would on-lend the funds to HPPEA in local currency (LE). EEA would repay the loan in LE, with the Central Bank bearing the exchange risk.

*b. Investment Bank of Egypt (IBE)*
If the local currency component is to be financed locally, loans can be received from the Investment Bank of Egypt and other local banks. The Investment Bank of Egypt provides finance at 14.5% interest with 2 years grace period and 12 years repayment. These conditions are fixed by the Government. Other commercial local banks offer similar conditions.

In principle, a 2 year grace period for a project with a construction period of 5 years implies that repayment has to start already before commissioning. This would require refinancing of the loan during construction; and if the banks do not agree to this, financing of the project may be at risk. In this particular case, however, the 2 year grace period for local loans does not constitute a problem. The local component consists largely in installation and erection of the foreign equipment which does not take place until the years -2 and -1, that is the local funds are required only in the last two years of construction.

Table 10.7 summarizes potential sources of funds for the HPPEA/EEA component and the respective financing conditions.

Table 10.7: SOURCES OF FUNDS AND LENDING CONITIONS

| Financier | Borrower | Currency | Interest Rate % | Commitm. Fee % | Grace Period | Repaym. Period |
|---|---|---|---|---|---|---|
| KfW | Central Bank | DM | 0.75 | | 10 | 30 |
| | on-lending to EEA | DM | 6.0 | 0 75 | Constr. | 20 |
| KfW | on-lending to EEA | LE | 6.0 | 0.75 | Constr. | 20 |
| Investment Bank of Egypt | EEA | LE | 14.5% | - | 2 | 12 |

10.6.2 Financing Plan

As part of the cashflow analysis a financing plan for HPPEA/EEA's cost share has been set up, based on the relevant investment in local and foreign currencies and financing conditions as described above. Interest during construction (IDC) and commitment fees add to the total financing requirements. Both IDC and fees depend on the specific loan mix and, therefore, are different for alternative financing options.

Financing is tailored to the financing requirements: capital requirements and financing exactly match each other, so that the net cashflow is 0 in each year during the construction period. Excess cash would unnecessarily increase financing costs in later years, and a negative cashflow would indicate that the project cannot meet its financial obligations.

Since the foreign component of the equipment costs to be borne by HPPEA/EEA, even including price contingencies, are below KfW's budget limitations, no additional funding for the foreign component is required. KfW will provide 100% of the required loan amount including IDC, which amounts to 104.49 million US$ (156.7 million DM).

For the local component (36.17 million US$ before IDC and fees), two alternatives are considered:

Financing Option 1: KfW also finances the local component, with the exception of the customs duties (12% of the local component) which is financed by HPPEA/EEA with a loan from the Investment Bank of Egypt.

Financing Option 2: The entire local component is financed by HPPEA/EEA with a loan from the Investment Bank of Egypt.

Table 10 8 summarizes total financing requirements for both financing options. It is obvious that Financing Option 2 - due to the higher interest rate of the local loan which increases interest during construction - requires more funds than Financing Option 1.

Table 10.8: FINANCING REQUIREMENTS FOR ALTERNATIVE FINANCING OPTIONS

| | % of Capital Expenditure (Capex) | | | Total Requirements (Current Prices) | | | |
|---|---|---|---|---|---|---|---|
| | | | | $10^6$ US$ | | | $10^6$ LE |
| | LC | FE | Total | LC | FE | Total | Total |
| Equity Financing | 0% | 0% | 0% | - | - | - | - |
| Debt Financing | 100% | 100% | 100% | 36.2 | 91.9 | 128.1 | 487.6 |
| Total Capex | 100% | 100% | 100% | 36.2 | 91.9 | 128.1 | 487.6 |
| **Financing Option 1** | | | | | | | |
| IDC and Fees | 11% | 14% | 13% | 3.7 | 12.6 | 16.3 | 62.8 |
| Total Requirements | 111% | 114% | 113% | 39.9 | 104.5 | 144.4 | 550.5 |
| **Financing Option 2** | | | | | | | |
| IDC and Fees | 22% | 14% | 16% | 7.8 | 12.6 | 20.4 | 78.9 |
| Total Requirements | 122% | 114% | 116% | 44.0 | 104.5 | 148.5 | 566.5 |

The contribution of the individual loans to total debt financing in both financing options is shown in Table 10.9. Financing Options 1 and 2 are identical with regard to the foreign component, but differ with regard to the local component: in Financing Option 1 the major part of the local component is financed by KfW at an interest rate of 6%, while in Financing Option 2 the entire local component is financed locally at an interest rate of 14.5%. Due to the higher interest rate of the local loan, interest during construction for the local component - and as a consequence total financing requirements - are considerably higher in Financing Option 2 than in Option 1.

Table 10.9: FINANCING OPTIONS

| | | Financing Option 1 | | | Financing Option 2 | |
|---|---|---|---|---|---|---|
| | | Foreign Component | Local Component | | Foreign Component | Local Component |
| Borrower | | KfW | KfW | Investm. Bank of Egypt | KfW | Investm. Bank of Egypt |
| Currency | | DM | LE | LE | DM | LE |
| % of Foreign Debt | | 100% | | | 100% | |
| % of Local Debt | | | 88% | 12% (duties) | | 100% |
| Interest Rate | | 6% | 6% | 14.5% | 6% | 14.5% |
| Commitment Fee | | 0.75% | 0.75% | - | 0.75% | - |
| Grace Period (Years) | | 4 | 2 | 2 | 4 | 2 |
| Repayment Period (Years) | | 21 | 20 | 12 | 21 | 12 |
| Loan Amount | $10^6$US$ | 91.9 | 32.0 | 4.2 | 91.9 | 36.2 |
| (excluding IDC) | $10^6$DM[1)] | 137.9 | 48.0 | 6.3 | 137.9 | 54.3 |
| Loan Amount | $10^6$US$ | 104.5 | 34.8 | 5.1 | 104.5 | 44.0 |
| (including IDC) | $10^6$DM | 156.7 | 52.2 | 7.7 | 156.7 | 66.0 |

[1)] Exchange rate DM/US$: 1.5

10.6.3 Cashflow Projections

In order to assess the risk that EEA's revenues from energy sales of the project may not suffice to service the loans, the project cashflow has been set up for the two alternative financing options with sensitivity cases. The cashflow comprises loan disbursements and revenues from electricity sales as cash inflows, and investment costs, operating expenses and financing costs as cash outflows. As the cashflow statement

deals only with cash transactions, non-cash items, such as depreciation, bad debt write-offs, intangibles and others do not appear on it.

Since the share to be financed by HPPEA/EEA consists only of M&E equipment with an assumed service life of 30 years, the cashflow is set up only for this period. A period of thirty years is sufficient to assess the project impact on EEA's financial performance, as all loans will have been repaid by the end of this period, and further projections would not provide any information which can be used in this assessment.

The cashflow projections are based on the following:

*Investment Costs*
Investment costs comprise only the costs for the equipment necessary for hydropower generation, including duties and physical and price contingencies, see last column of Table 10 4. Allowances for engineering during construction are not included in these cost items. The effect of a 10% increase in initial investment cost on the project cashflow is tested in the sensitivity analysis.

*Financing*
Loan disbursements and financing costs are derived from Financing Options 1 and 2 as described above.

*Operating Expenses*
Operating and maintenance costs are estimated as a percentage of the initial investment (1.5% of the equipment costs including price contingencies and engineering) This percentage is based on worldwide experience, but has been checked against the actual operation and maintenance cost of Aswan II powerplant (270 MW installed capacity) for the fiscal years 1990/91 to 1993/94 For a powerplant of the size of the High Aswan Dam (2.100 MW) O&M costs are considerably lower in relation to the initial investment costs; however, for Aswan II the actual expenses (9.47 million LE - excluding depreciation - in 1993/94) were found to be consistent with the assumed percentages.

O&M costs are assumed to increase with local inflation. For working capital, particularly at the start of commercial operation, it is assumed that this will be contributed from the liquidity of EEA.

*Revenue*
The revenue projection is based on sales of the annual energy production of 462 6 GWh at Naga Hammadi Substation and an average unit revenue at the relevant voltage level of 9.35 pta/kWh in 1996 (for justification see Section 10.2.3). Under the assumption that tariffs are increased annually in line with inflation (without any further real increases), the average unit revenue will have increased from 9.35 pta/kWh in 1996 to 16.58 pta/kWh by the time the powerplant goes into operation in 2006. This compares to a required unit revenue for covering O&M costs and financing costs in 2006 of 15.41 pta/kWh under Financing Option 1 and 19.47 pta/kWh under Financing Option 2.

Since the level of the average unit revenue determines the liquidity situation of the project, two further tariff scenarios are considered, one assuming a 25% increase in real terms (equivalent to the required increase from 9.35 pta/kWh to reach the LRMC level of 11.76 pta/kWh on the relevant distribution level - 12.0 pta/kWh minus 2%) distributed over 5 years from 1997 to 2001; and the other assuming increases below the inflation rate for the next 3 years and equivalent to the inflation rate thereafter. Table 10.10 summarizes the three scenarios considered.

deals only with cash transactions. non-cash items. such as depreciation. bad debt write-offs. intangibles and others do not appear on it.

Since the share to be financed by HPPEA/EEA consists only of M&E equipment with an assumed service life of 30 years. the cashflow is set up only for this period. A period of thirty years is sufficient to assess the project impact on EEA's financial performance, as all loans will have been repaid by the end of this period, and further projections would not provide any information which can be used in this assessment.

The cashflow projections are based on the following:

*Investment Costs*
Investment costs comprise only the costs for the equipment necessary for hydropower generation, including duties and physical and price contingencies, see last column of Table 10 4. Allowances for engineering during construction are not included in these cost items. The effect of a 10% increase in initial investment cost on the project cashflow is tested in the sensitivity analysis.

*Financing*
Loan disbursements and financing costs are derived from Financing Options 1 and 2 as described above.

*Operating Expenses*
Operating and maintenance costs are estimated as a percentage of the initial investment (1.5% of the equipment costs including price contingencies and engineering) This percentage is based on worldwide experience, but has been checked against the actual operation and maintenance cost of Aswan II powerplant (270 MW installed capacity) for the fiscal years 1990/91 to 1993/94 For a powerplant of the size of the High Aswan Dam (2,100 MW) O&M costs are considerably lower in relation to the initial investment costs; however, for Aswan II the actual expenses (9.47 million LE - excluding depreciation - in 1993/94) were found to be consistent with the assumed percentages.

O&M costs are assumed to increase with local inflation. For working capital, particularly at the start of commercial operation, it is assumed that this will be contributed from the liquidity of EEA.

*Revenue*
The revenue projection is based on sales of the annual energy production of 462.6 GWh at Naga Hammadi Substation and an average unit revenue at the relevant voltage level of 9.35 pta/kWh in 1996 (for justification see Section 10.2.3). Under the assumption that tariffs are increased annually in line with inflation (without any further real increases), the average unit revenue will have increased from 9.35 pta/kWh in 1996 to 16.58 pta/kWh by the time the powerplant goes into operation in 2006. This compares to a required unit revenue for covering O&M costs and financing costs in 2006 of 15.41 pta/kWh under Financing Option 1 and 19.47 pta/kWh under Financing Option 2.

Since the level of the average unit revenue determines the liquidity situation of the project, two further tariff scenarios are considered, one assuming a 25% increase in real terms (equivalent to the required increase from 9.35 pta/kWh to reach the LRMC level of 11.76 pta/kWh on the relevant distribution level - 12.0 pta/kWh minus 2%) distributed over 5 years from 1997 to 2001; and the other assuming increases below the inflation rate for the next 3 years and equivalent to the inflation rate thereafter. Table 10.10 summarizes the three scenarios considered.

Table 10.10: TARIFF SCENARIOS

| | Assumption | Tariff in 1996 pta/kWh | Tariff in 2006 pta/kWh |
|---|---|---|---|
| Tariff Scenario 1 | Increased with local inflation | 9 35 | 16 58 |
| Tariff Scenario 2 | Increased with inflation plus real increase of 25% (distributed over a period of 5 years) to reach LRMC level | 9 35 | 20 55 |
| Tariff Scenario 3 | Increased at 50%, 65%, 80% of local inflation over 3 years, at 100% thereafter | 9 35 | 15 32 |

*Income Taxes*

At present no income taxes are to be paid, but this situation may have changed by the time operations begin, as by then the electricity subsector may have been privatized or at least corporatized. Income taxes would then probably be recovered through raising the consumer tariffs, so that taxes would not have a direct impact on EEA's revenue.

*Development of Inflation and Foreign Exchange Rate*

As discussed above, international prices are assumed to increase by 2 5% annually and domestic prices by 8% until 1999 and by 5% thereafter. The LE/US$ exchange rate will remain constant at 3 40 until 1999; thereafter the LE will be devalued in line with the inflation differential.

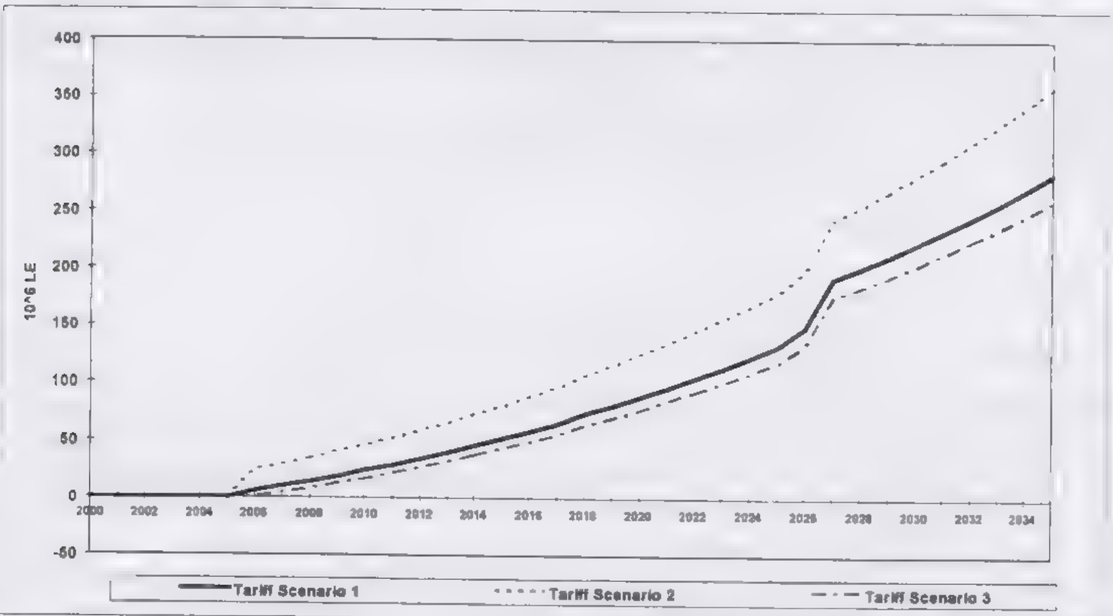
The cashflows for Financing Options 1 and 2 (both for Tariff Scenario 1) are presented in Appendix Y4. Figure 10-6 shows the effect of alternative tariff scenarios on the project cashflow for Financing Option 1. As can be seen from the figure:

- The project can be financed under the Financing Option 1 and Tariff Scenario 1 without incurring any liquidity problems. The net cashflow, i.e. the balance of cash inflows and cash outflows, will always be positive. Interest payments on the outstanding loan balance decrease over time, thus reducing financing costs and increasing the net cashflow. After complete repayment of the KfW loans in the year 2025 the net cashflow will increase considerably.

- Even if the average sales tariff is increased at a rate below the local inflation rate for the next three years (Tariff Scenario 3), revenues will be sufficient to cover O&M and financing costs.

- If the tariff is increased to LRMC level (Tariff Scenario 2), revenues will exceed O&M and financing costs by far, thus enabling EEA to build up considerable reserves.

Figure 10-6: PROJECT CASHFLOW (FINANCING OPTION 1) FOR ALTERNATIVE TARIFF SCENARIOS




Financing Option 2 involves higher financing costs, due to the higher interest rate and shorter repayment period of the local loan. Figure 10-7 compares the project cashflow of Financing Option 2 for alternative tariff scenarios. The figure shows the following:

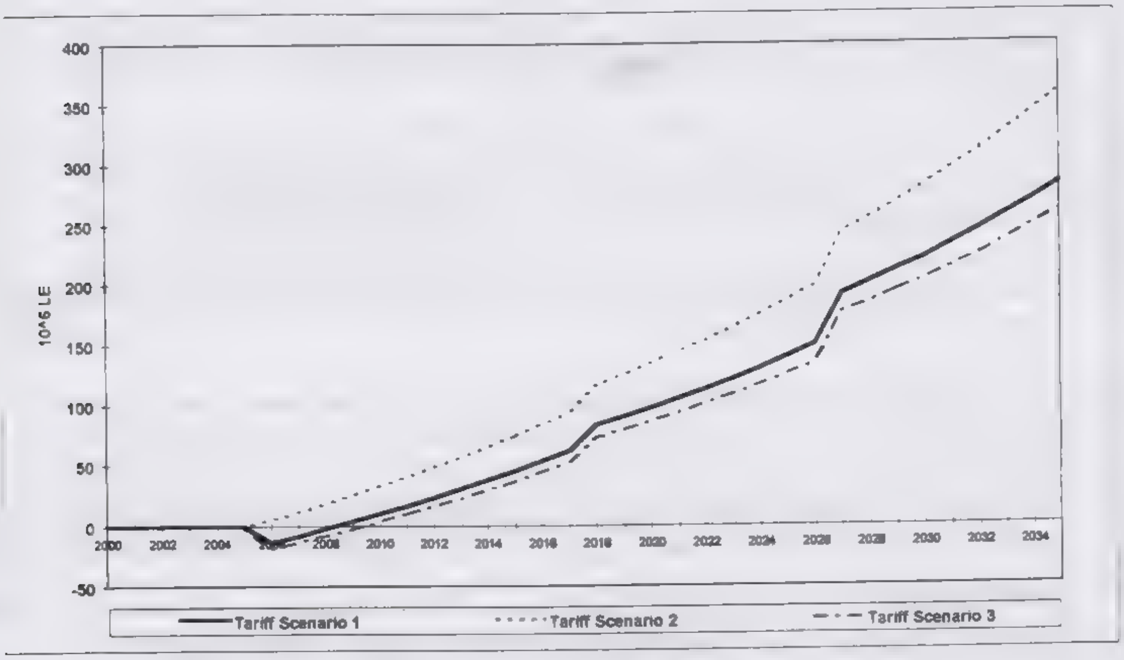
- If under Financing Option 2 the tariff is kept constant in real terms and only increased in line with inflation (Tariff Scenario 1), revenues are not sufficient to cover O&M costs and financing costs for the first three years of operation. Already in the year 2009 the net cashflow is positive again. The cashflow increases considerably after repayment of the local loan in 2017 and the foreign loan in 2025.

- The liquidity problems in the first three years can be solved by increasing tariffs to LRMC level (Tariff Scenario 2), or by extending the repayment period for the local loans.

- Under the assumptions of Tariff Scenario 3 (initial tariff increase below the inflation rate) the net cashflow is negative for the first four years.

- Any liquidity problems in the early years of operation under Tariff Scenarios 1 and 3 are compensated by the rapid increase in the net cashflow in later years.

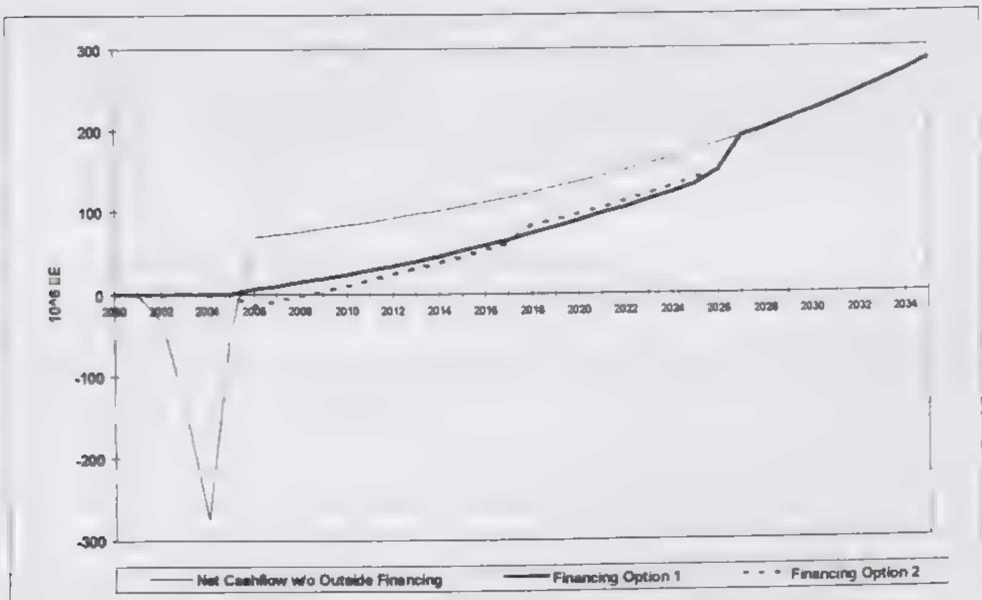
Figure 10-7: PROJECT CASHFLOW (FINANCING OPTION 2) FOR ALTERNATIVE TARIFF SCENARIOS




Financing Options 1 and 2 are compared in Figure 10-8 (for Tariff Scenario 1). While for Financing Option 1 the net cashflow is always positive, it is negative in 2006, 2007, and 2008 for Financing Option 2, as described above. After 2017, when the local loan has been repaid, the net cashflow for Financing Option 2 is above the net cashflow for Option 1. When all loans have been repaid, the cashflows of both options are identical.

For comparison, Figure 10-8 also shows the cashflow for the hypothetical case without outside financing, i.e. the cashflow as it would develop under the assumption that the investment is financed from EEA's internal sources. In this case no financing costs would have to be paid, so that the net cashflow (revenues minus O&M costs) is much higher than for Financing Options 1 and 2 during their repayment periods.

Figure 10-8: PROJECT CASHFLOW FOR FINANCING OPTIONS 1 AND 2 (TARIFF SCENARIO 1)



The effect of changes in the investment cost has been tested in a sensitivity analysis which leads to the following results:

- If investment costs are 10% above cost estimates, the project has minor liquidity problems in the first year of operation, but in principle can still be financed under Financing Option 1 (with Tariff Scenario 1).

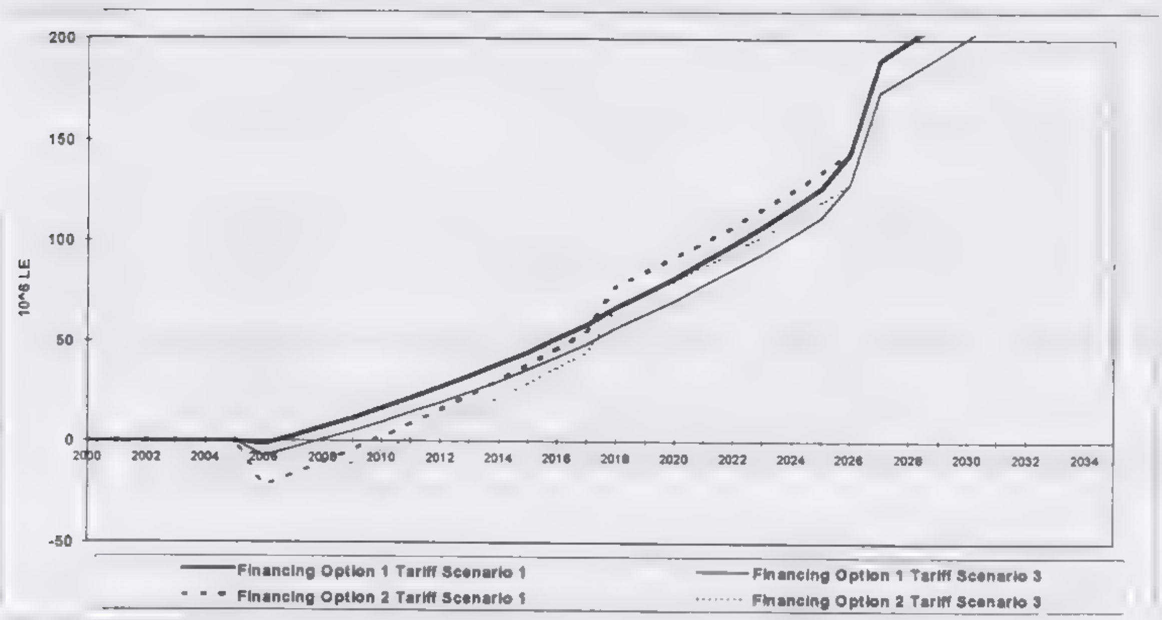
- For Financing Option 2, financing conditions will have to be renegotiated to avoid liquidity problems. Otherwise the project will have liquidity problems for the first four years.

- Under Tariff Scenario 3, the net cashflow will be negative for two years for Financing Option 1, and for five years for Financing Option 2.

These results are shown in Figure 10-9 (which for reasons of clarity focus on the years up to the end of the repayment period).

If investment costs are 10% below cost estimates, EEA's financial situation will profit considerably from the implementation of the project. Under Financing Option 2 the Project would only have minor liquidity problems in the first year.

Figure 10-9: PROJECT CASHFLOW FOR INVESTMENT COST CHANGES



## 11. SUMMARY AND CONCLUSIONS

This Report presents the results of the third stage in the successive refinement of the Naga Hammadi Barrage Development Study from its conception through to the detail of a full Feasibility Study. The earlier stages comprised a Conceptual Study in 1993, followed by an Interim Study in 1994.

The primary objective of the Naga Hammadi Barrage Development Study is to formulate the most appropriate scheme ensuring continued supply of water to the irrigation areas downstream of the existing Barrage. At the same time it should utilize the natural resource of the River Nile for the purpose of hydropower generation. Doing nothing to the Barrage, now 65 years old, would result in a progressive reduction of its structural and operational safety, placing at risk the continuing agricultural production of the irrigation areas. Current estimates of this annual production in economic terms is around $1.700 \times 10^6$ LE ($500 \times 10^6$ US$).

During the Conceptual Study, a detailed appraisal of the condition of the existing Naga Hammadi Barrage was undertaken with the aim to investigate if the Barrage could be rehabilitated to ensure its structural integrity for long term and safe continuation of the irrigation supply. Simultaneously, the development of a New Barrage was also assessed. Hydropower generation was investigated at both the existing Barrage and the New Barrage.

Insufficient information to accurately determine the local geologic and geotechnical conditions of the site precluded the Conceptual Study from making a definitive statement about the technical viability of the alternatives proposed. In order to refine the geotechnical data, the Interim Study became a prerequisite to the Feasibility Study itself.

Investigations and engineering evaluations from the Interim Study resulted in the following:

- Even if costly structural and foundation improvements of the existing Barrage would be implemented, there remains an unquantifiable risk in the reliability of some of the basic measures of rehabilitation with resultant uncertainty in the duration of the extended service life. Rehabilitation was therefore not seen as an acceptable alternative to construction of a New Barrage which could also incorporate a hydropower development.

- With the prevailing geological conditions a New Barrage can only be constructed at reasonable cost and using proven construction methods at a site where a continuous, impermeable layer exists at a shallow depth beneath the site. Without this layer the construction pit cannot be maintained in a dry condition and there would be an ever-present danger of piping. These requirements for the temporary establishment of a large construction pit in the River Nile were only encountered some 3.5 km downstream of the existing Barrage.

Following on from the results of the Interim Study, the work for the Feasibility Study was directed at carrying out further field investigations at the site of the New Barrage, optimizing the layout, and advancing it to full feasibility level. In order to quantify the merits of including hydropower in a New Barrage, it was necessary to consider two layout options to a comparable level of detail, namely:

- A layout entailing a New Barrage with Hydropower,
- A layout entailing a New Barrage without Hydropower (Base Case).

The "Base Case", thus defined for the sole purpose of guaranteeing continued irrigation, allows cost separation of the hydropower component from the New Barrage itself. This enables direct definition of the economic performance of the hydropower component.

The site investigations for the New Barrage location have confirmed continuity of the clay layer at a level of some 15 m asl, approximately 45 m below the present riverbed level. Engineering evaluations for the New Barrage have also confirmed the site selection in the existing river channel to the west of Geziret El Dom. Optimization of the construction procedure resulted in a single construction pit for the main

concrete structures of the Barrage (hydropower plant, sluiceway and navigation lock), accessible from Geziret El Dom Diversion during construction would be by a separate diversion canal on the west bank of the River Nile. The diversion canal will be closed by an embankment dam at the end of the construction period.

Based on a redefining of the original Terms of Reference through the Aide Memoire (MOPWWR, MEE and KfW), the headpond level of the New Barrage was set at 65 9 m asl throughout the year, and the operation of the hydropower plant was determined to be run-of-river Its operation is therefore independent of the daily load demands of the electricity supply system (UPS).

The composition of the layout comprises a navigation lock with a chamber area 170 m x 17.0 m on the right side, a sluiceway with 7 gate openings, each 17 m wide, in the river channel and a hydropower plant with 4 bulb turbine units, 16 0 MW each, to the left. River discharge up to 1,840 $m^3$/s can pass through the powerplant, the sluiceway activated only for higher river flow and floods

The flood release capability of the sluiceway permits evacuation of the 7,000 $m^3$/s emergency release from the HAD with a headpond surcharge of some 1.25 m. During evacuation of the 1 10,000 year flood release from the HAD, there will be no rise in headpond level. For additional safety, the navigation lock is designed to participate in the evacuation of extreme flood discharges.

The capacity of the hydropower component was optimized by incremental generation cost analysis, resulting in an installed capacity of 64 MW and a total average energy generation of 463.7 GWh.

By re-establishing a headpond level of 65.9 m asl, comparable to the design headpond for the existing Barrage, the water level would be increased above present operating levels by 0.8 m in the low flow season and 0.5 m in the high flow season. This, coupled with the construction and implementation of the New Barrage, will result in project-related impacts for which an Environmental Impact Assessment was undertaken. This identified and quantified the impacts from which an Environmental Mitigation programme was developed. For the headpond-related impacts, the effects can be classified as those associated with the river reach between the existing and New Barrages and the moderate river level increases upstream, while the construction-related effects are limited to the near vicinity of the Barrage site.

Headpond-related effects were thoroughly assessed by mathematical modelling of river and groundwater levels for existing and future conditions, on the basis of a substantial amount of field data. This highlighted the present unsatisfactory condition of a significant area of the main open drainage system. The MOPWWR intends to remedy this situation prior to project implementation. Therefore, a sustainable open drain maintenance programme was developed in the study as a prerequisite to assess future impacts of the project. Mitigation of headpond-related imapcts then mainly includes expansion and upgrading of the tile drainage network in selected areas, installation of sanitation systems and improvement of housing foundations to prevent damp, again in selected areas which would otherwise be affected by the project. The majority of the limited upstream effects can be mitigated by physical improvements, only those related to the loss land by inundation of river islands and some possible impact on agricultural production in small areas remain.

The benefits of a raised headpond level, however, are also significant in terms of reduced net pumping cost for irrigation which are some 3.6 GWh/year. In addition, investment and energy pumping savings will accrue since the new irrigation areas being developed downstream between the New Barrage and Assiut can be supplied from the headpond instead by pumping from the downstream river reach.

Within the construction area, the land acquired for construction activities will be compensated land-by-land by reclaiming areas in the floodway at Geziret El Dom with excess excavation material. Overall there will be a surplus of some 114.2 feddan of perrenial agricultural land. There will be no resettlement of households as a result of the project, rather new houses will be constructed on Dom Island. The sociological impacts are expected to be minimal. The EMP provides for full compensation for lost income from agricultural production and wages during the period of construction.

The costs before shadow pricing of the New Barrage development and the reference cases for evaluation of hydropower and losses/gains incurred from environmental effects and their mitigation (expressed in financial terms) are presented below at the 1995 price level.

Table 11.1: INVESTMENT COSTS (Sep. 1995 Level)

| | New Barrage | | Base Case | | Hydro Power Component | | M&E Equipment | |
|---|---|---|---|---|---|---|---|---|
| | $10^6$ US$ | $10^6$ LE | $10^6$ US$ | $10^6$ LE | $10^6$ US$ | $10^6$ LE | $10^6$ US$ | $10^6$ LE |
| A: Civil Works | 146.3 | 497.5 | 112.8 | 383.7 | 33.5 | 113.8 | 0.0 | 0.0 |
| B: Hydrom. Equipment | 28.8 | 97.9 | 21.3 | 72.3 | 7.5 | 25.5 | 0.0 | 0.0 |
| C: Mechan. Equipment | 36.6 | 124.3 | 0.0 | 0.0 | 36.6 | 124.3 | 36.6 | 124.3 |
| D: Electrical Equipment | 50.0 | 170.0 | 1.6 | 5.4 | 48.4 | 164.5 | 50.0 | 170.0 |
| Physical Contingencies | 33.2 | 112.8 | 19.7 | 66.9 | 13.5 | 45.9 | 8.7 | 29.5 |
| Engineering | 28.0 | 95.2 | 14.8 | 50.2 | 13.2 | 45.1 | 0.0 | 0.0 |
| Subtotal | 322.9 | 1097.7 | 170.2 | 578.5 | 152.7 | 519.2 | 95.2 | 323.8 |
| Environmental Cost | 13.8 | 46.8 | 13.4 | 45 4 | 0.4 | 1.2 | 0.0 | 0.0 |
| Total Investment (constant terms) | 336.7 | 1144.5 | 183.6 | 623.9 | 153.1 | 520.4 | 95.2 | 323.8 |

Differential costs between the New Barrage and New Barrage without Hydropower were assigned for the economic evaluation of hydropower, assuming that the minimum cost at which continued irrigation could safely be assured would be that of the New Barrage without Hydropower. Investment required for continued irrigation is regarded as a basic necessity, economically justified by the high net value of annual production from all irrigation areas supplied.

For the Hydropower Component, economic benefits depend largely on the fuel price for alternative thermal generation which was assumed from a recently completed LRMC approach on natural gas. to the year 2013, and thereafter by pricing of gas at 85% of the international oil price. The need for additional energy and capacity in the UPS was justified by an electricity demand growth rate of 6% p.a. and EEA's system expansion plans which show insufficient capacity in 2006. The hydropower plant would have the objective to add firm capacity of some 41.8MW to the system and through non-firm energy production save fuel which would otherwise be generated by older thermal plant in the UPS.

Dynamic Production Cost calculated at 6% discount rate for the Hydropower Component are compared with those of alternative Thermal Generation in economic terms:

- Dynamic Production Cost Hydro 2.63 US$/KWh
- Dynamic Production Cost Thermal 2.82 US$/KWh

The advantage of electricity generation by the hydropower plant is further underlined by avoided $CO_2$ emissions of 272,000 tons/year.

By agreement between the MOPWWR and the MEE on June 26, 1996, the portion of the project to be financed by the EEA comprises the mechanical, electrical and generation equipment of the hydropower plant with the remaining costs to be borne by the MOPWWR.

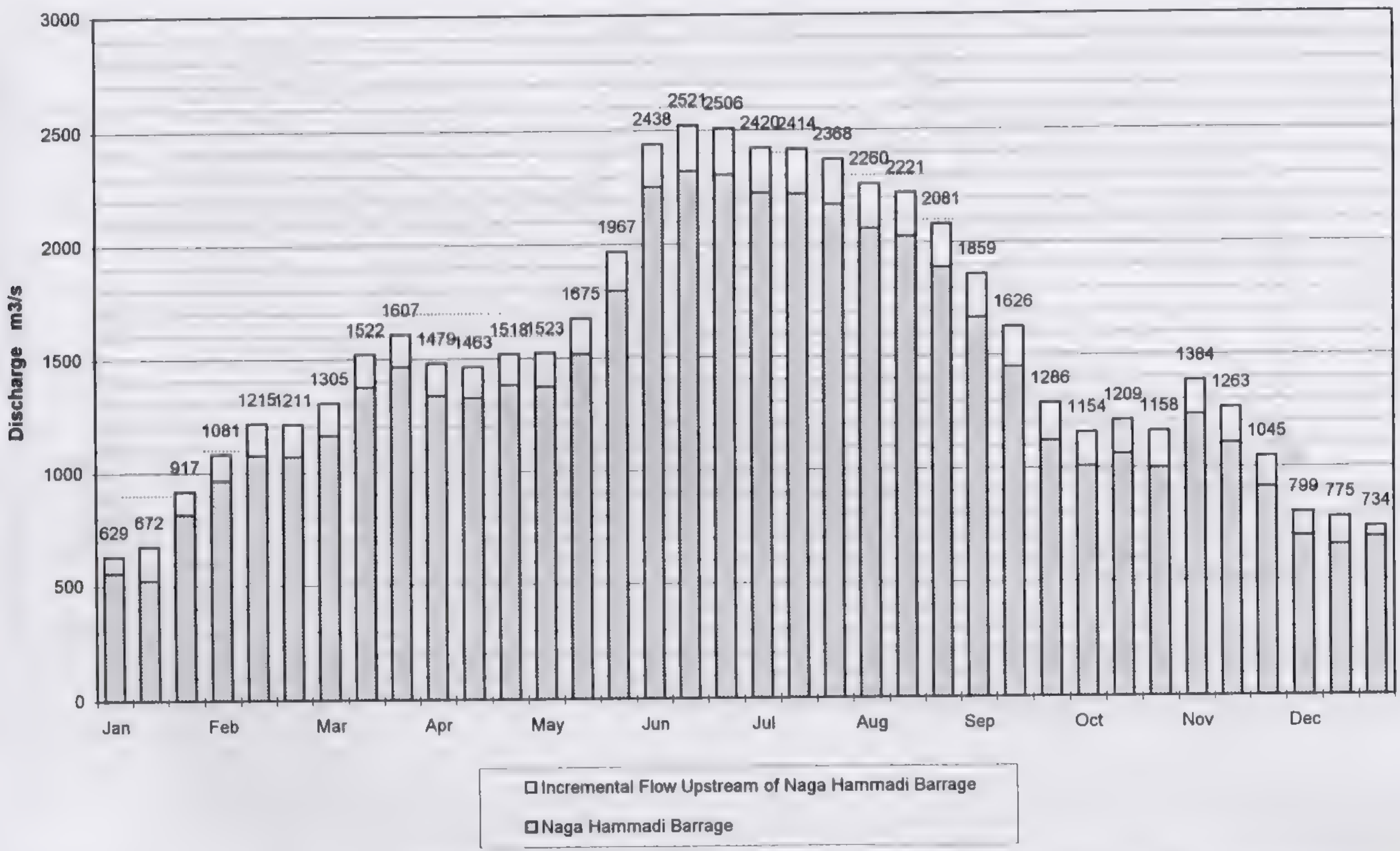
Capital expansion and operations in the irrigation sector are provided by the Government and are considered as nonrefundable state budget allocation. There are no fees for the use of irrigation water, and hence MOPWWR has no source of revenue to service loans.

Within the financial analysis, the Dynamic Production Cost of the hydropower plant was calculated and compared with the Dynamic Production Cost for thermal generation and the bulk supply tariff. The calculation of the DPC considering taxes and duties but no price contingencies, resulted in:

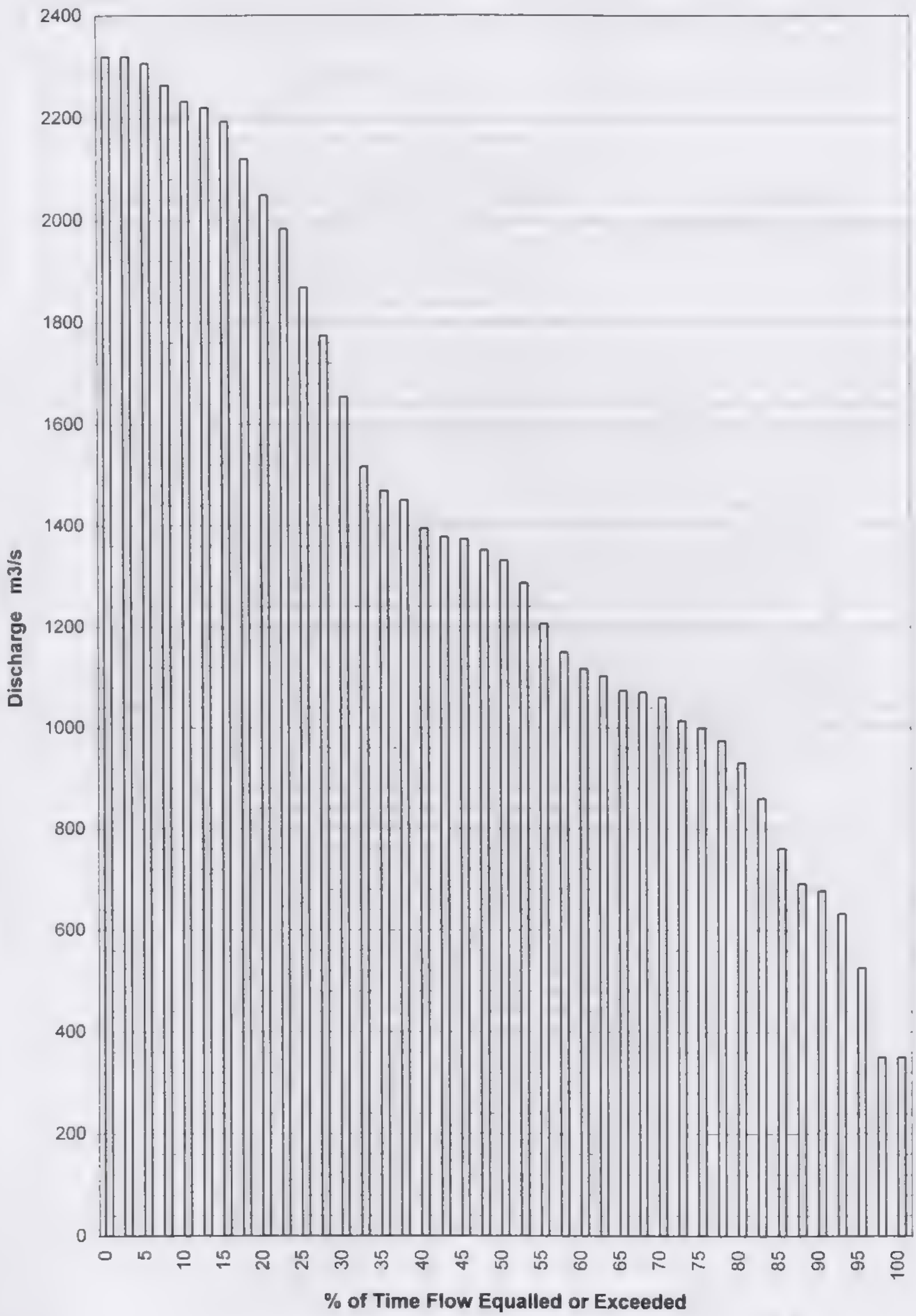
| Generation Plant | Hydropower | | Thermal | |
|---|---|---|---|---|
| Discount Rate | 5% | 8% | 5% | 8% |
| US cents/KWh | 1.85 | 2.46 | 1.89 | 2.08 |
| Pta/KWh | 6.28 | 8.37 | 6.44 | 7.09 |

The dynamic production cost of hydropower are slightly below those of thermal generation and far below present average unit revenue of about 9 35 pta/KWh. The DPC of thermal generation reflects the present fuel prices for thermal generation being far below world market level which puts hydropower generation at an unreasonable disadvantage.

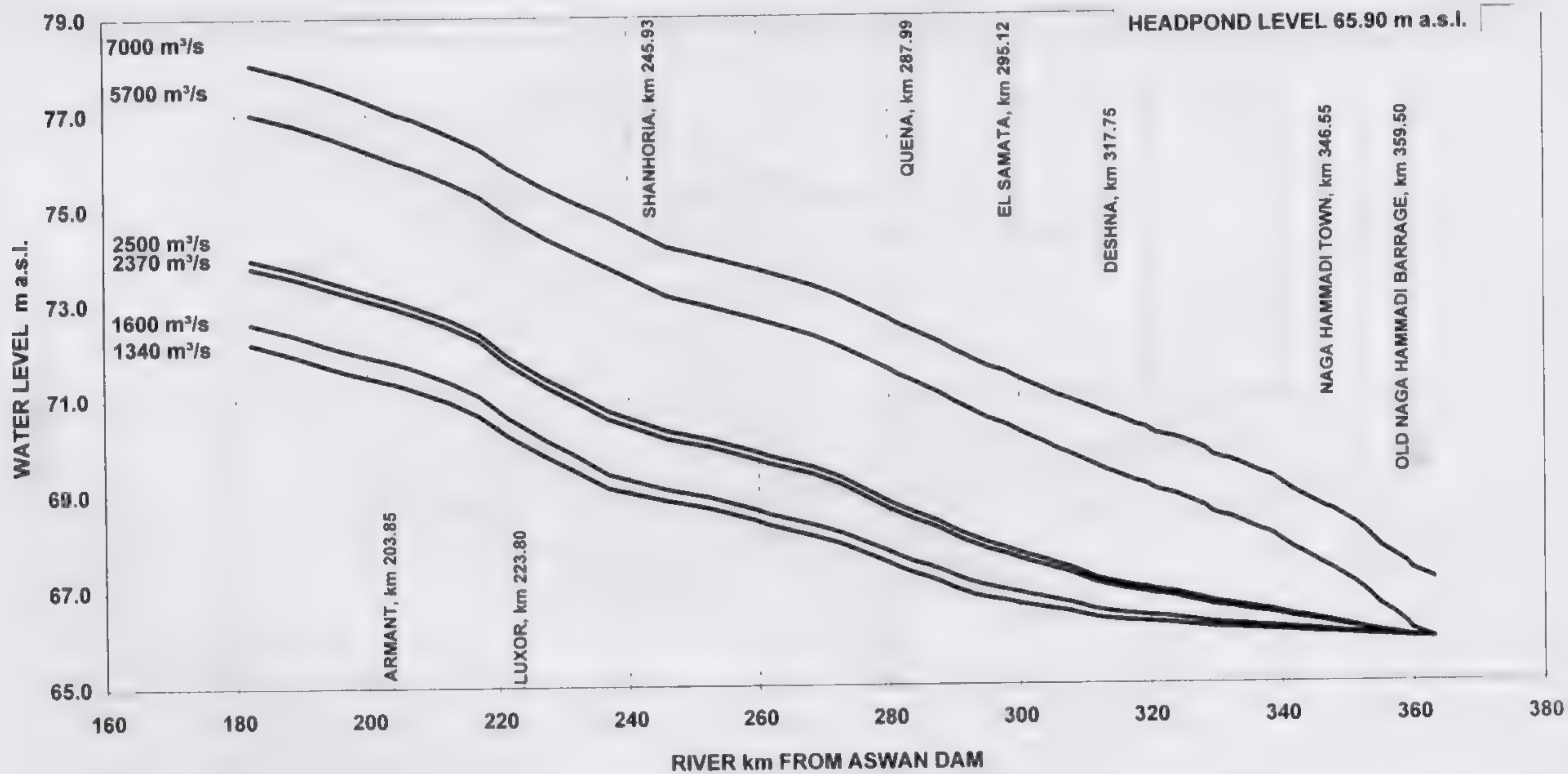
The cash flow analysis took into account EEA's present financial situation with the need for complete financing of the hydropower investment from external sources. Considering price contingencies on the investment side for different rates of inflation of foreign and local currencies and a tariff increase by local inflation, it is shown that the project can be financed without incurring any liquidity problems to EEA. Long-term, when loans are repaid, EEA's financial situation will considerably profit from the investment in the Naga Hammadi Hydropower Plant.



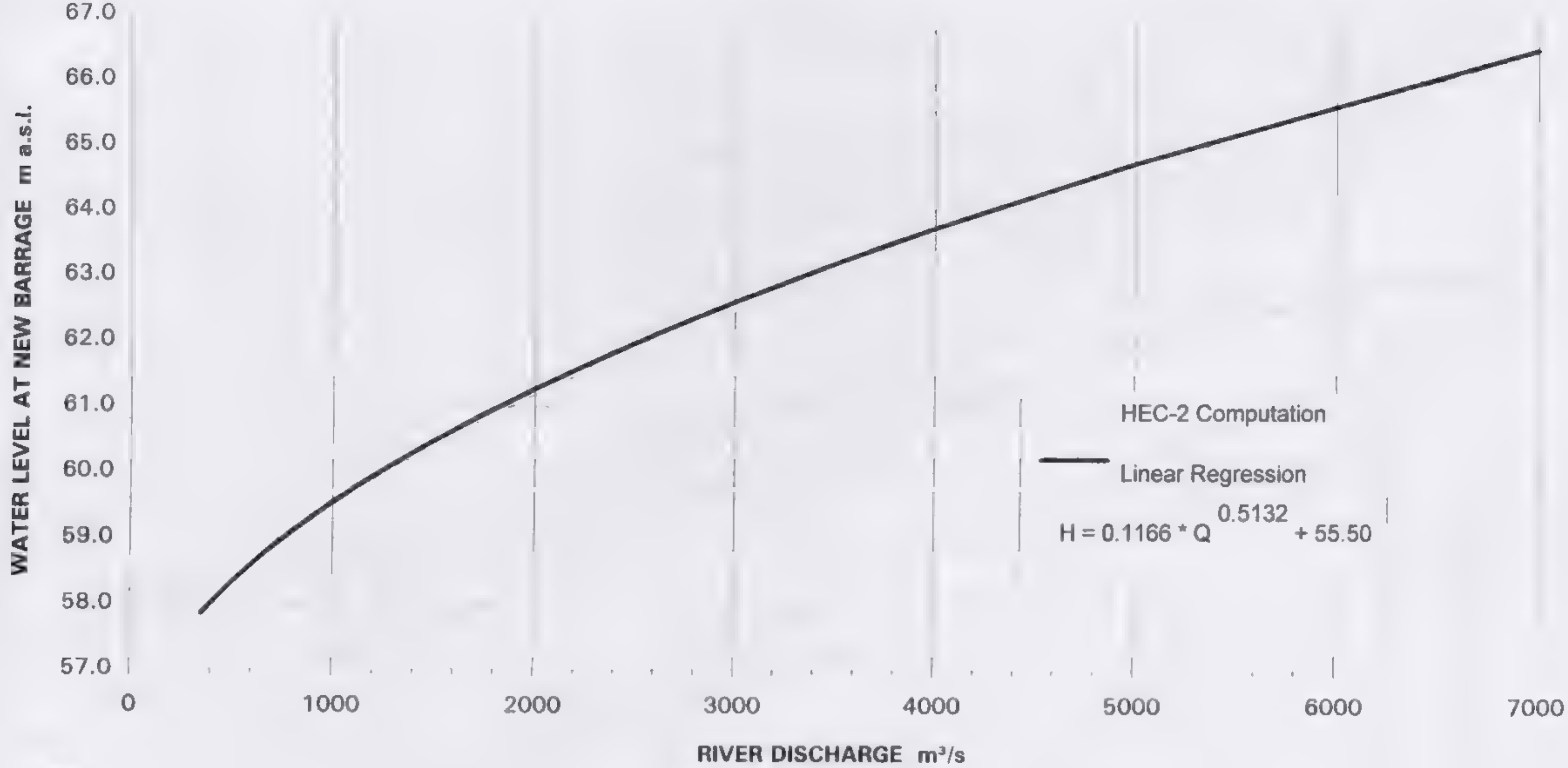

| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | AVERAGE FLOWS AT NAGA HAMMADI BARRAGE AND IN UPSTREAM REACH (1994 - 1996) | FIGURE 2-1 |



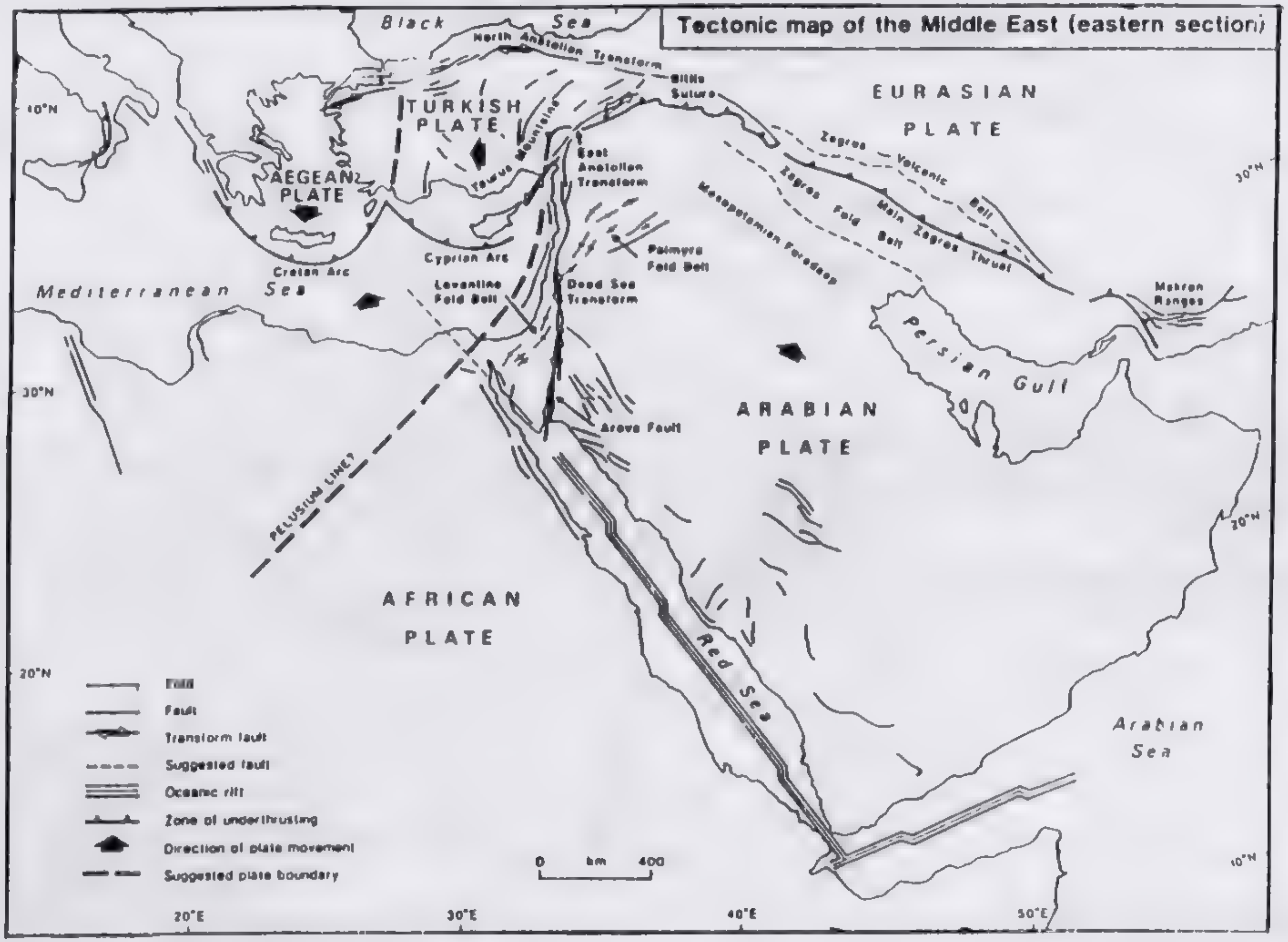

| | | | |
|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | NAGA HAMMADI BARRAGE DEVELOPMENT |
| Lahmeyer | Electrowatt | FEASIBILITY STUDY | JANUARY 1997 |
| Sogreah | A. C. E. | FLOW DURATION CURVE AT NAGA HAMMADI BARRAGE | FIGURE 2-2 |



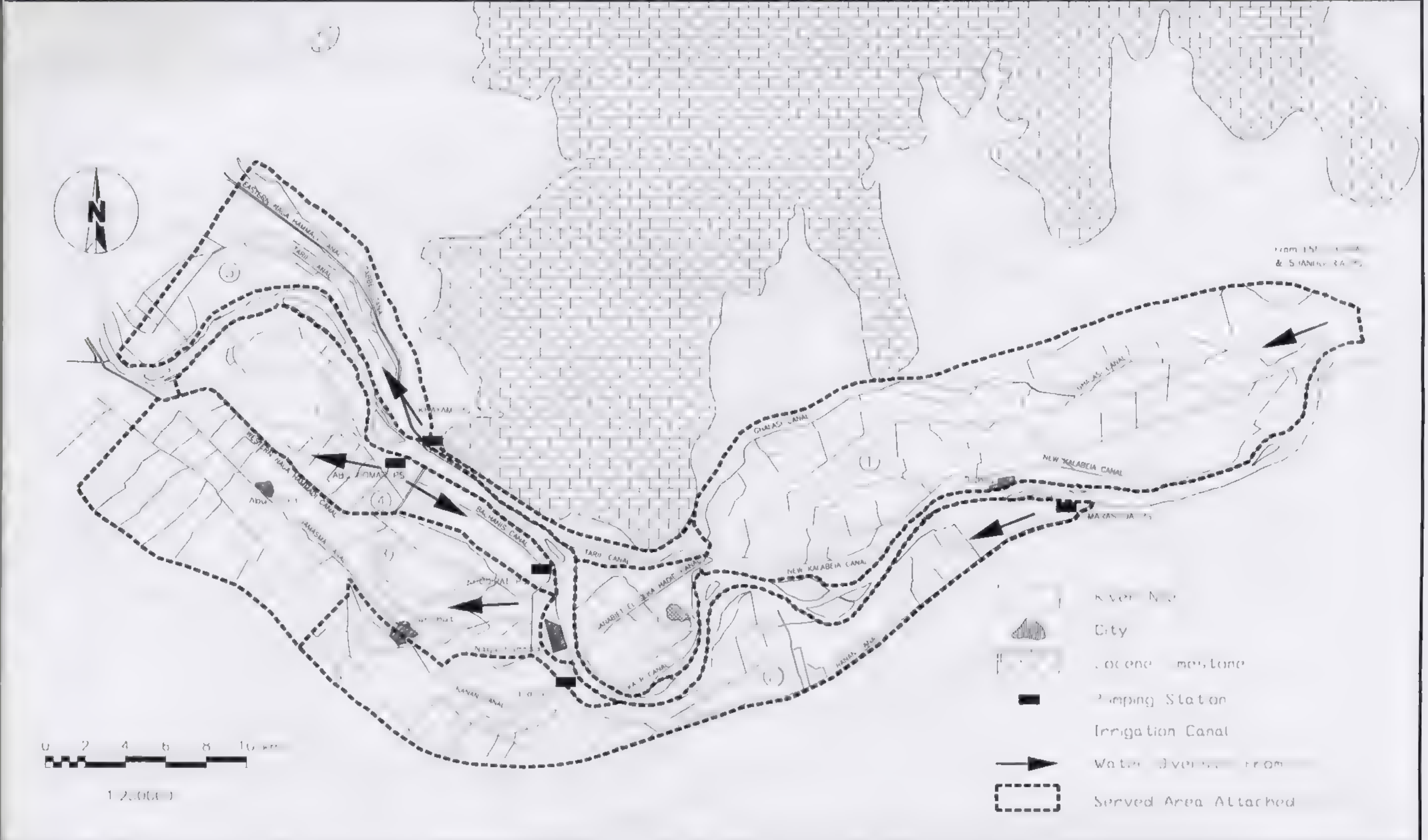
HEADPOND LEVEL 65.90 m a.s.l.
7000 m³/s
5700 m³/s
2500 m³/s
2370 m³/s
1600 m³/s
1340 m³/s
WATER LEVEL m a.s.l.
79.0
77.0
75.0
73.0
71.0
69.0
67.0
65.0
160
180
200
220
240
260
280
300
320
340
360
380
RIVER km FROM ASWAN DAM
ARMANT, km 203.85
LUXOR, km 223.80
SHANHORIA, km 245.93
QUENA, km 287.99
EL SAMATA, km 295.12
DESHNA, km 317.75
NAGA HAMMADI TOWN, km 346.55
OLD NAGA HAMMADI BARRAGE, km 359.50




| | | | | | |
|---|---|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | NEW BARRAGE TAILWATER RATING CURVE | FIGURE 2-4 |

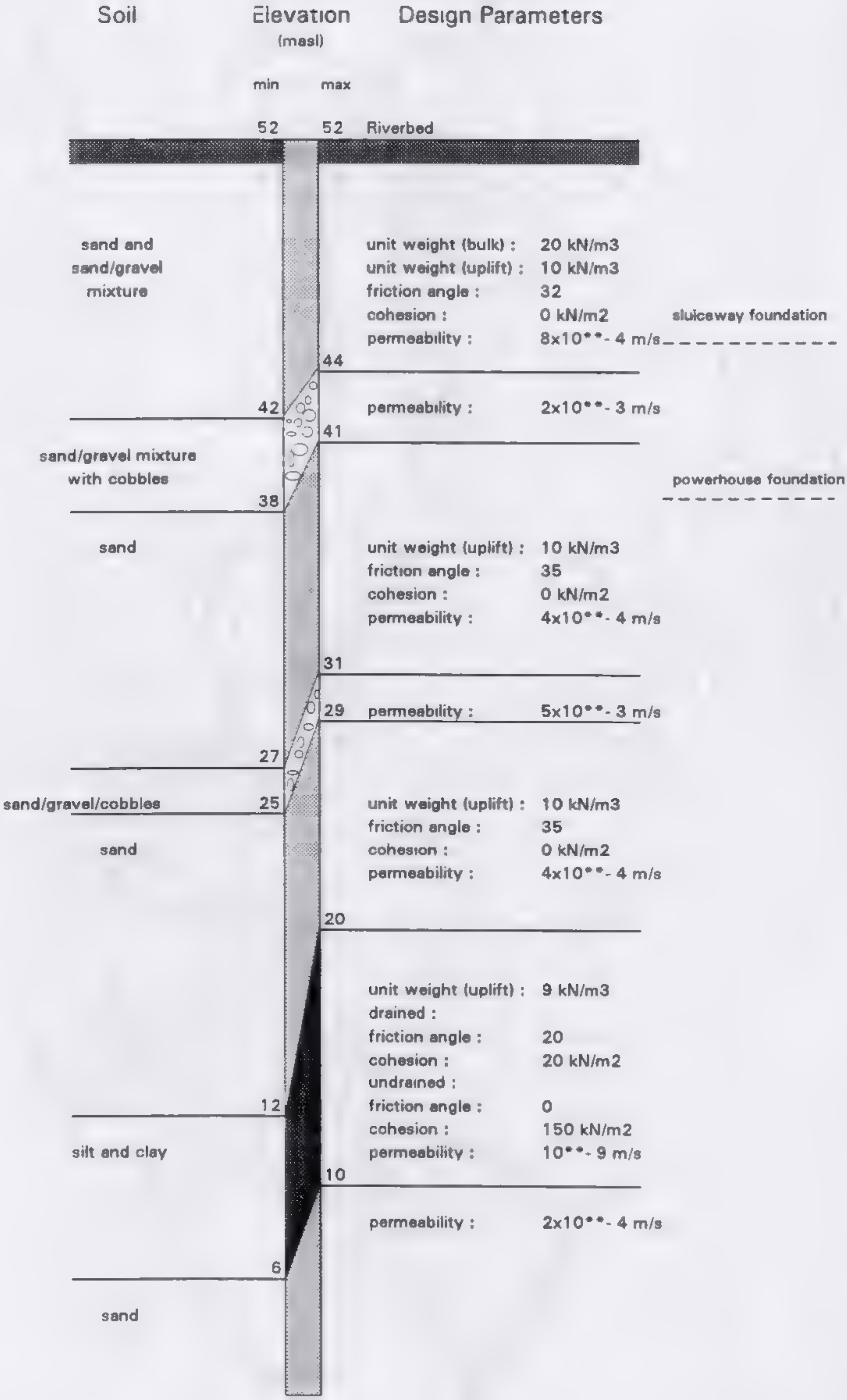



| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | AUGUST 1995 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | TECTONIC MAP OF MIDDLE EAST (E-SECTION) | FIGURE 2-5 |

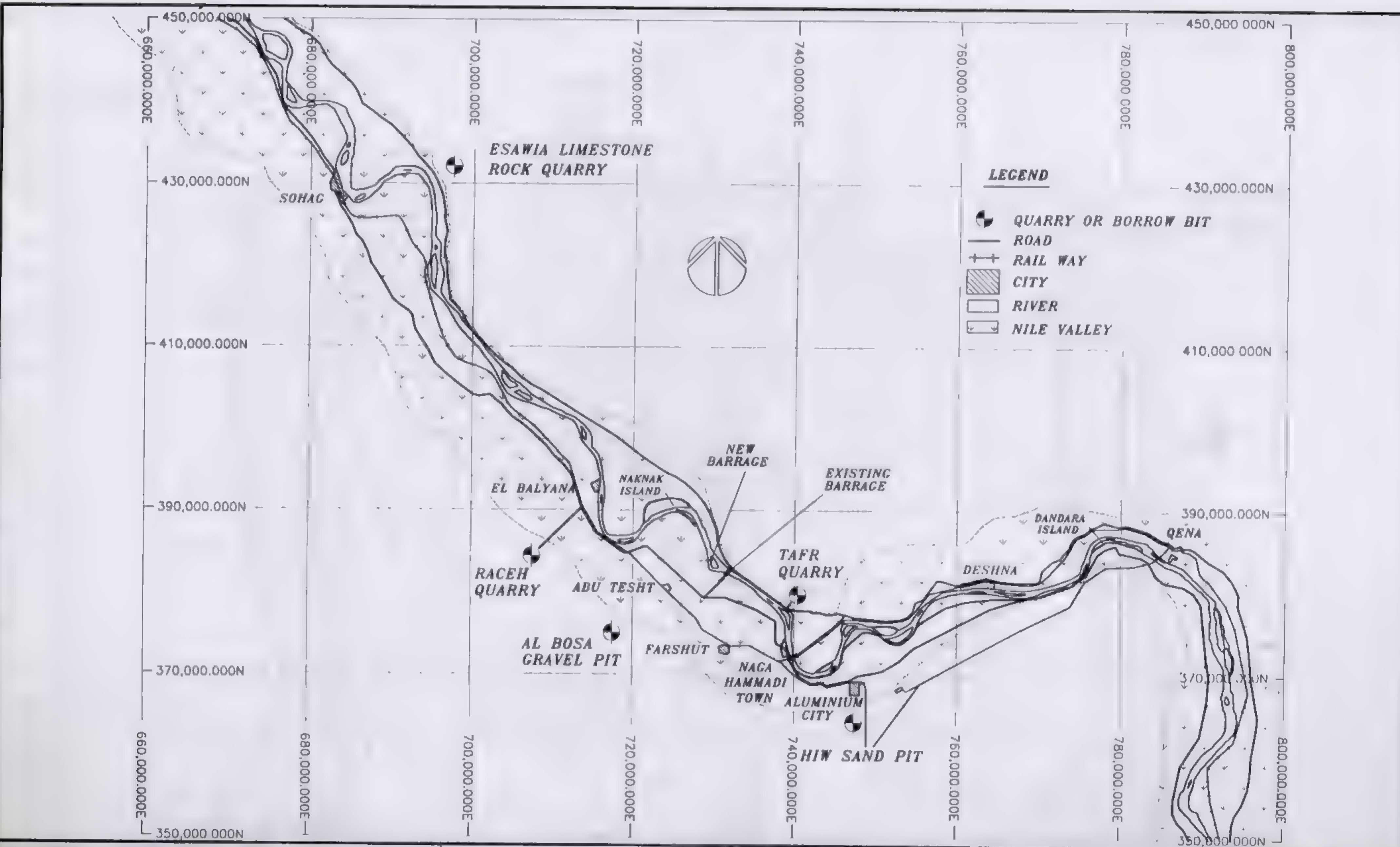



| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | EXTENSION OF IRRIGATION AREAS | FIGURE 2-6 |

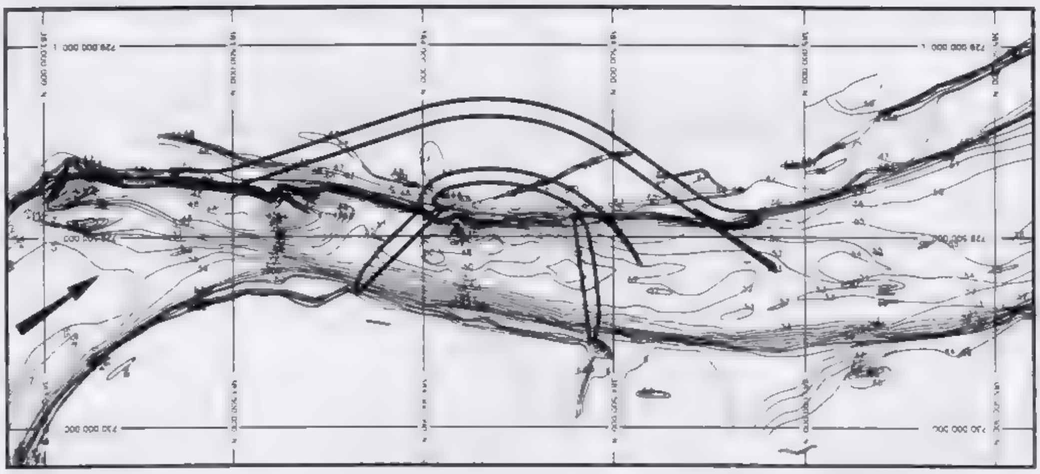

| | | | | | |
|---|---|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | INCREMENTAL AREAS WITH DEPTH OF GROUNDWATER CRITICAL TO AGRICULTURE | FIGURE 2-7 |



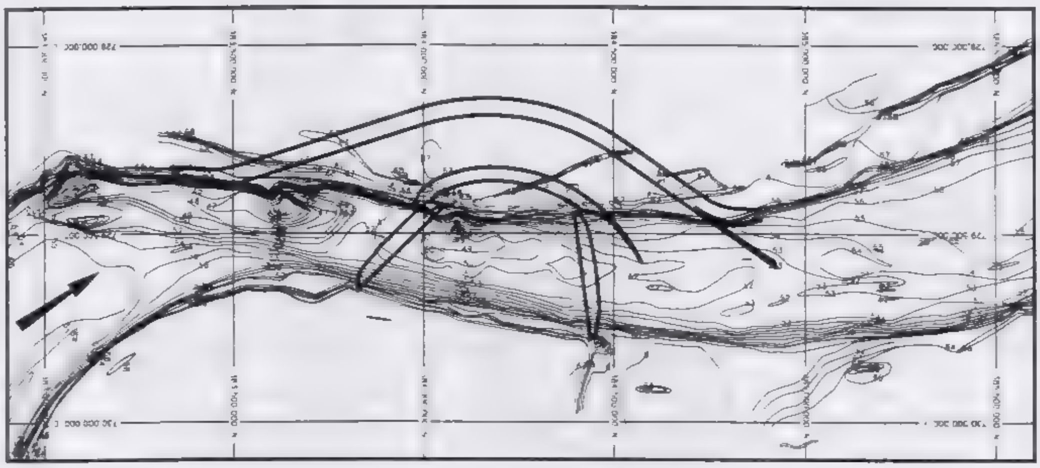

| | | | |
|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | NAGA HAMMADI BARRAGE DEVELOPMENT | |
| Lahmeyer | Electrowatt | FEASIBILITY STUDY | JANUARY 1997 |
| Sogreah | A. C. E. | GEOTECHNICAL DESIGN PARAMETERS - NEW BARRAGE | FIGURE 2-8 |



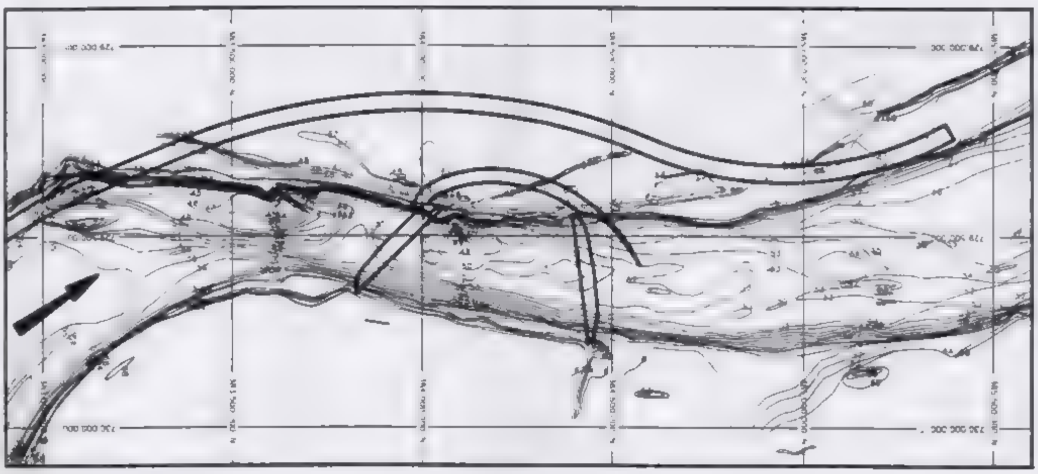

| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | LOCATION OF QUARRIES AND BORROW PITS | FIGURE 2-9 |



SHAPE 5

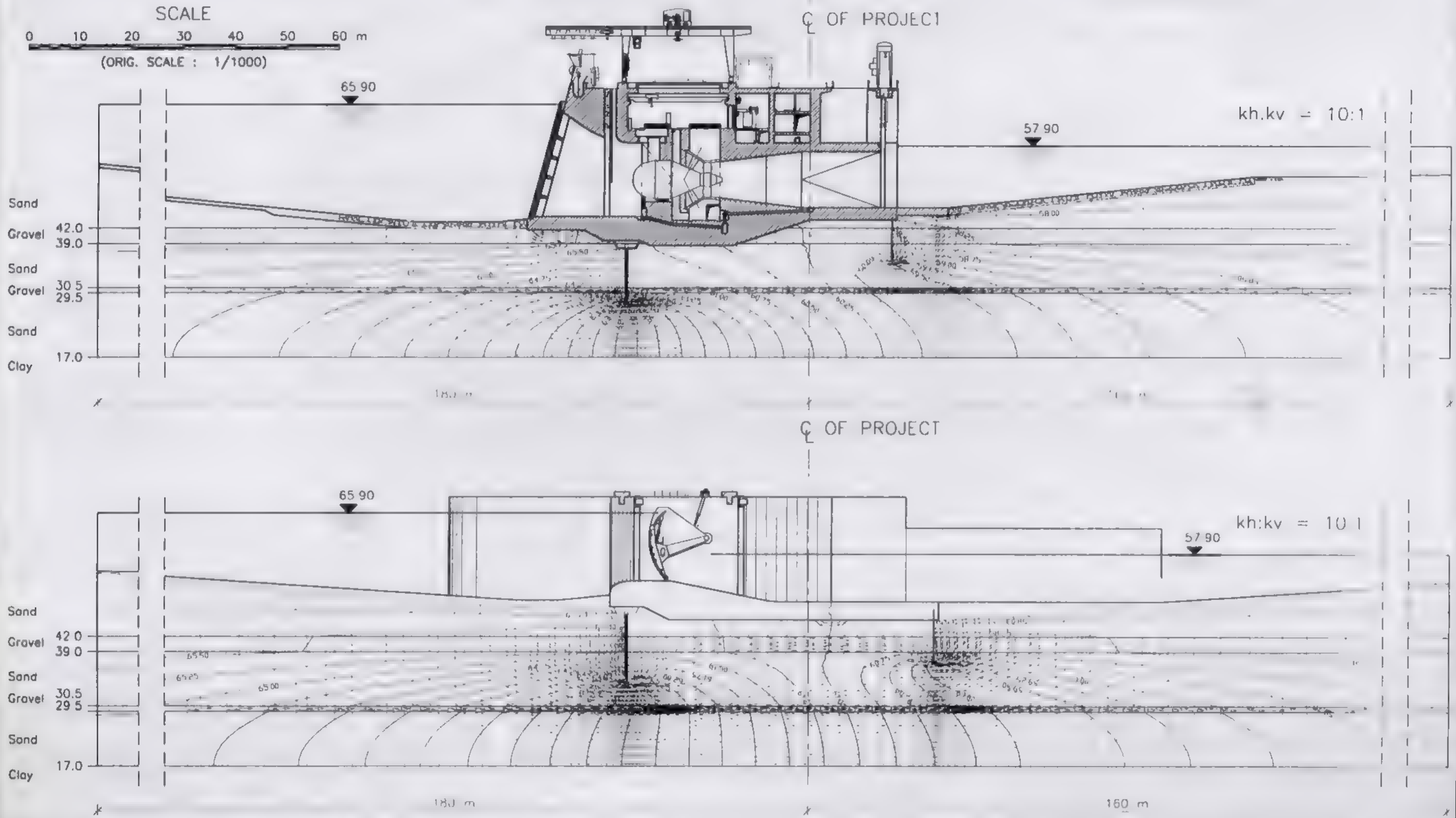


SHAPE 6

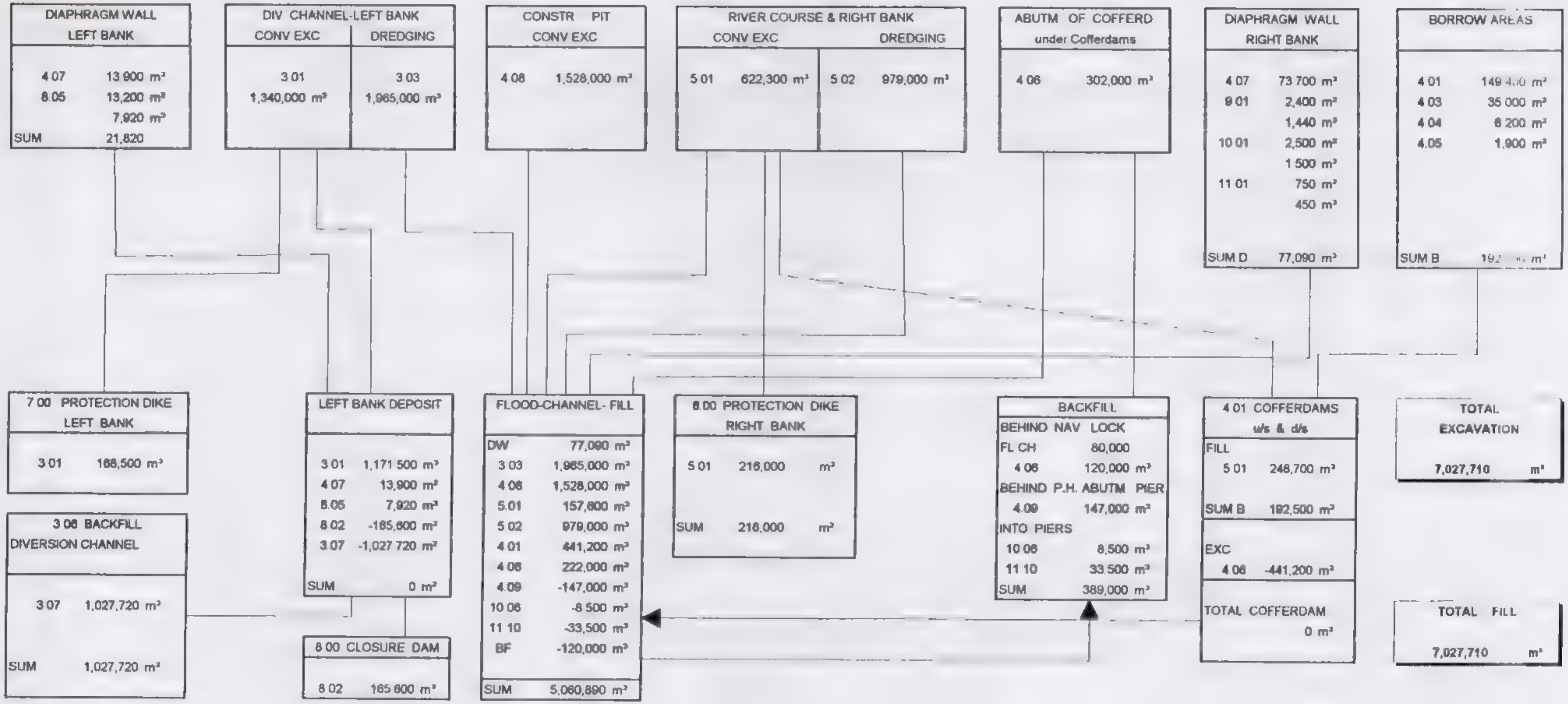


SHAPE 7

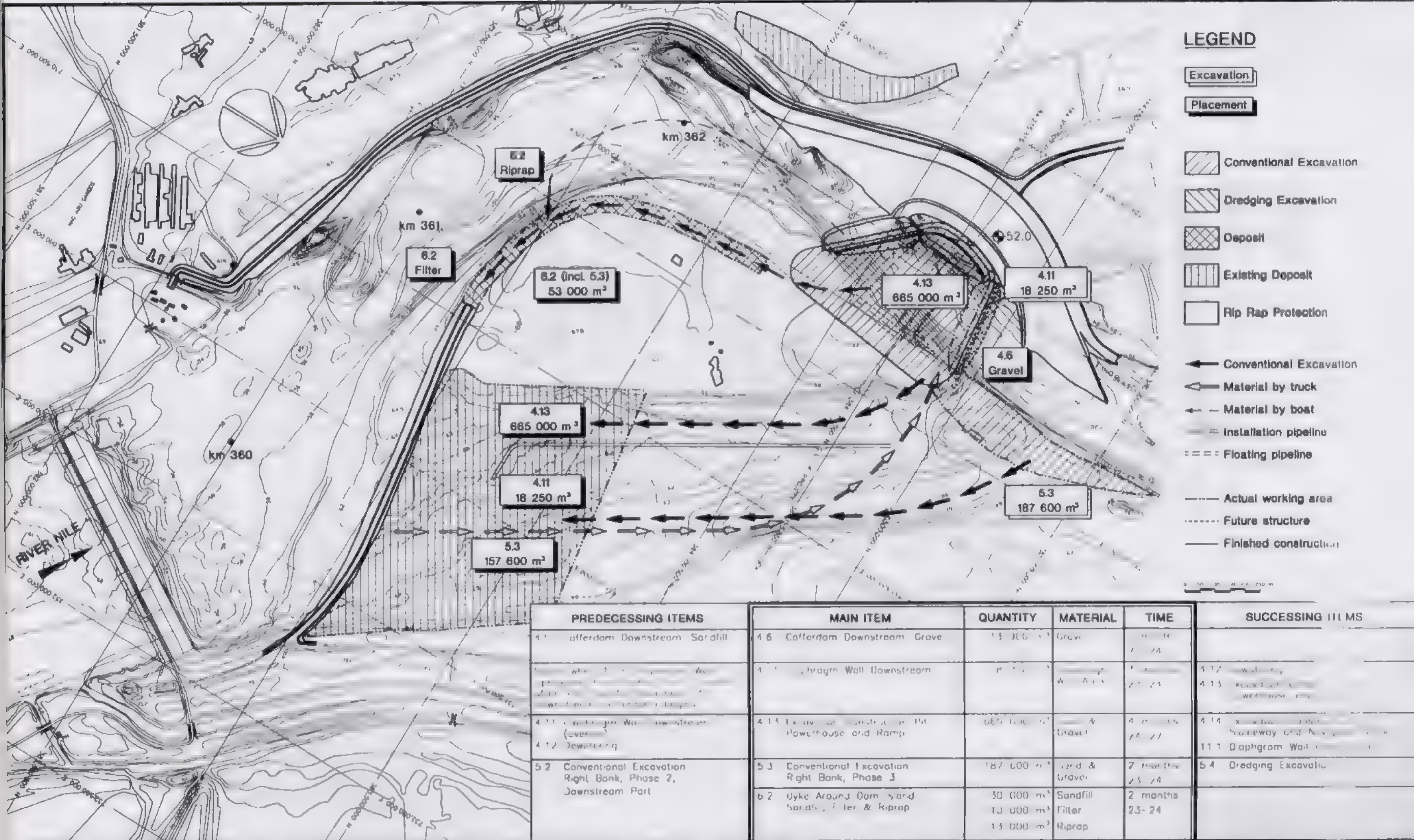
| | | | |
|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | NAGA HAMMADI BARRAGE DEVELOPMENT | |
| Lahmeyer | Electrowatt | FEASIBILITY STUDY | JANUARY 1997 |
| Sogreah | A. C. E. | SHAPES OF DIVERSION CANAL | FIGURE 4-1 |




| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | PIEZOMETRIC HEADS UNDER POWERHOUSE AND SLUICEWAY | FIGURE 4-2 |




| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | NEW BARRAGE - SCHEMATIC MASS FLOW DIAGRAM FOR EARTHWORKS | FIGURE 4-3 |




| PREDECESSING ITEMS | MAIN ITEM | QUANTITY | MATERIAL | TIME | SUCCESSING ITEMS |
|---|---|---|---|---|---|
| 4.[illegible] [illegible]offerdam Downstream Sandfill | 4.6 Cofferdam Downstream Gravel | 13 [illegible] | Gravel | [illegible] | |
| [illegible] | 4.[illegible] [illegible]hragm Wall Downstream | [illegible] | [illegible] | [illegible] | 4.12 [illegible]<br>4.13 [illegible] |
| 4.11 [illegible] Downstream [illegible]<br>4.12 Dewatering | 4.13 Excavation [illegible] Pit Powerhouse and Ramp | 66[illegible] | [illegible] & Gravel | [illegible] | 4.14 [illegible] Sluiceway and N[illegible]<br>11.1 Diaphgram Wall [illegible] |
| 5.2 Conventional Excavation Right Bank, Phase 2, Downstream Part | 5.3 Conventional Excavation Right Bank, Phase 3 | 187 600 m³ | Sand & Gravel | 7 months 23-24 | 5.4 Dredging Excavatio[illegible] |
| | 6.2 Dyke Around Dam Sand Sandfill, Filter & Riprap | 30 000 m³<br>13 000 m³<br>13 000 m³ | Sandfill<br>Filter<br>Riprap | 2 months 23-24 | |

MINISTRY OF PUBLIC WORKS AND WATER RESOURCES

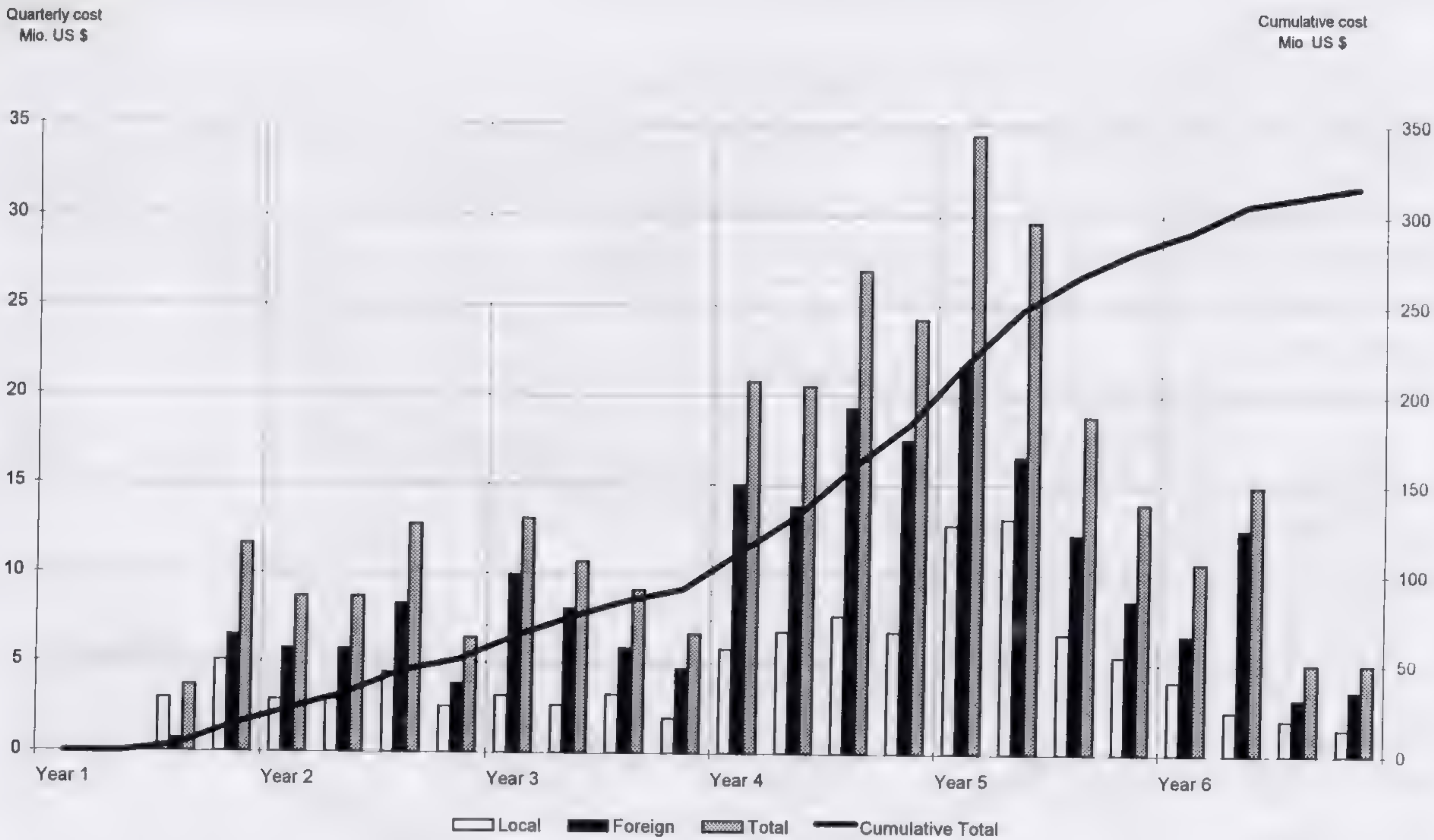
Lahmeyer | Electrowatt | Sogreah | A. C. E.

NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY

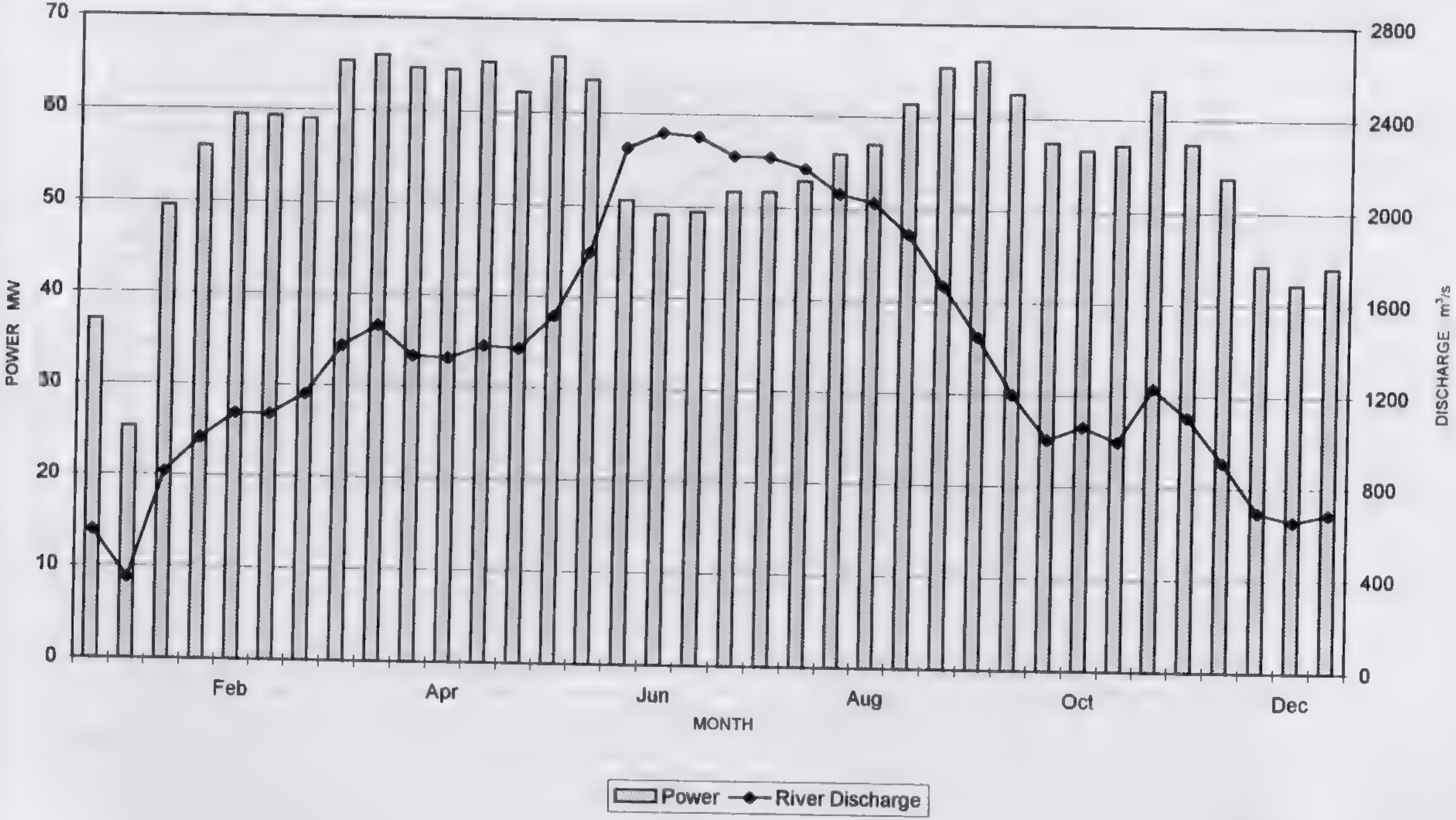
EXCAVATION OF CONSTRUCTION PIT AND BACKFILL OF FLOOD CHANNEL

JANUARY 1997

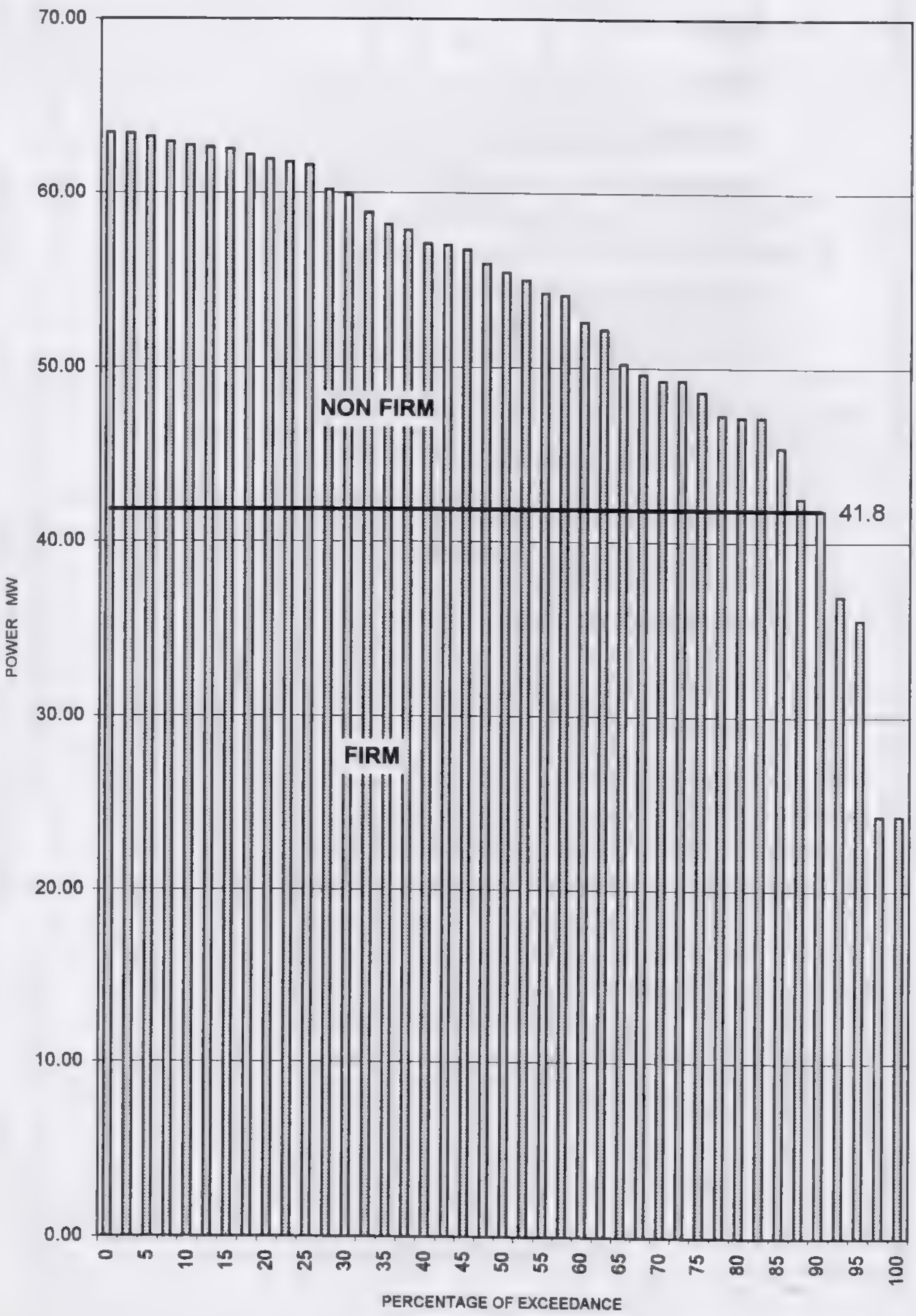
FIGURE 4-4



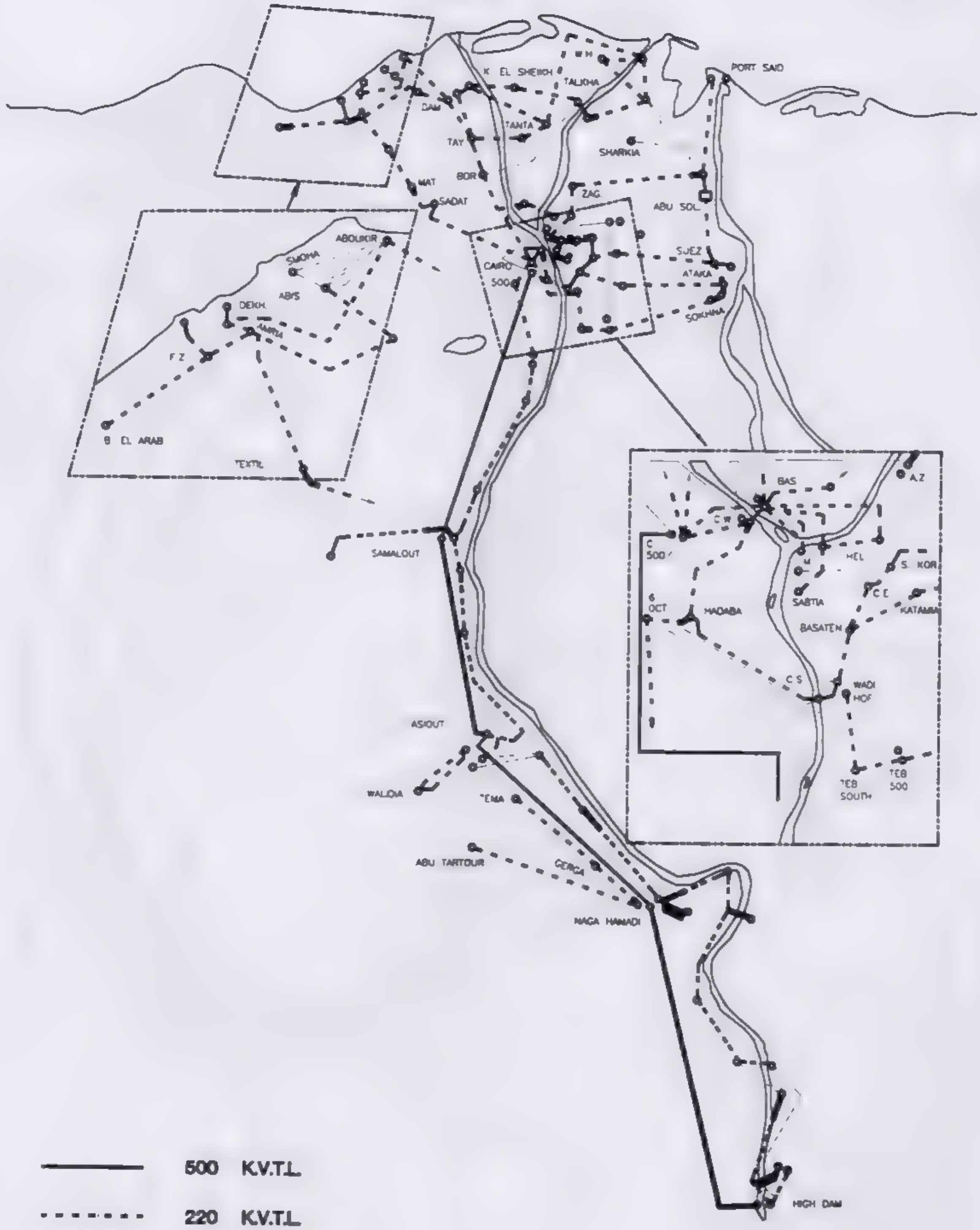

| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | NEW BARRAGE - CASH FLOW | FIGURE 4-5 |



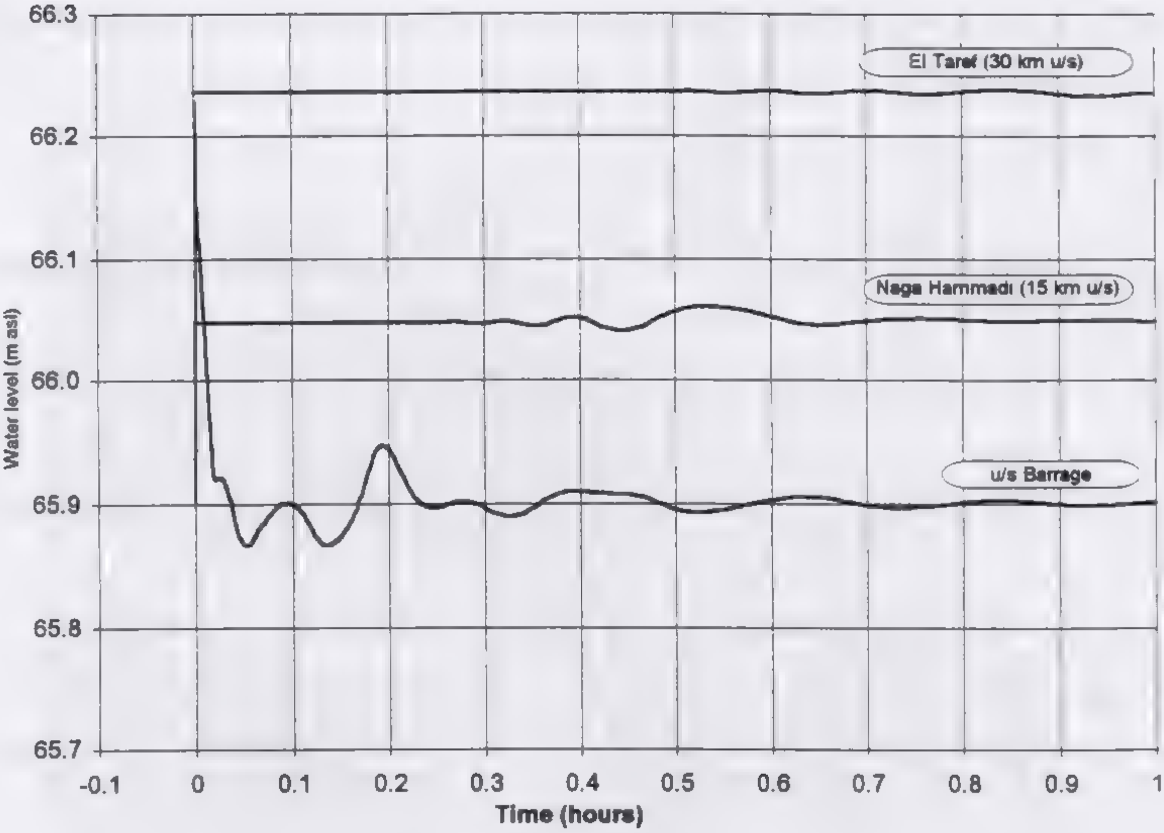

| | | | | | |
|---|---|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | DISTRIBUTION OF POWER GENERATION AT THE NEW BARRAGE | FIGURE 6-1 |

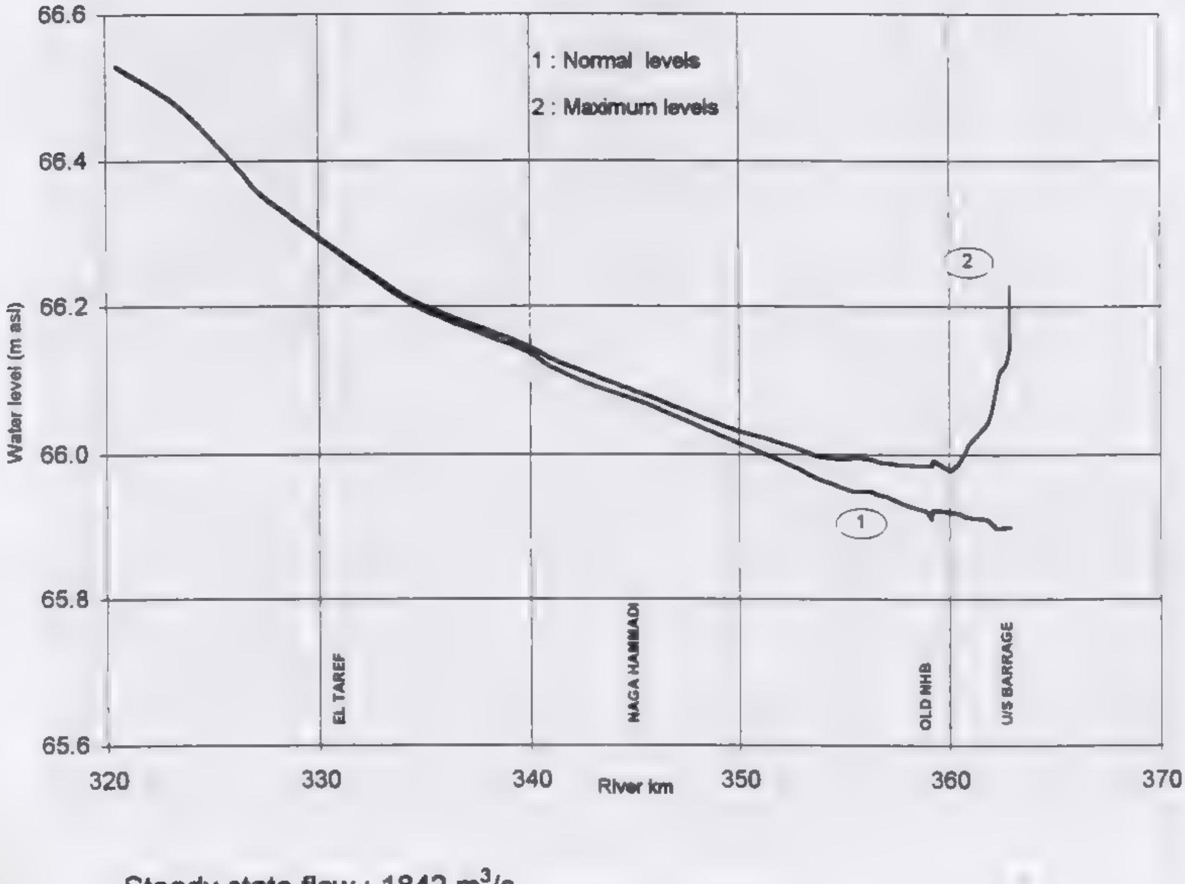



| | | | |
|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | NAGA HAMMADI BARRAGE DEVELOPMENT |
| Lahmeyer | Electrowatt | FEASIBILITY STUDY | JANUARY 1997 |
| Sogreah | A. C. E. | DURATION CURVE OF POWER GENERATION AT THE NEW BARRAGE | FIGURE 6-2 |



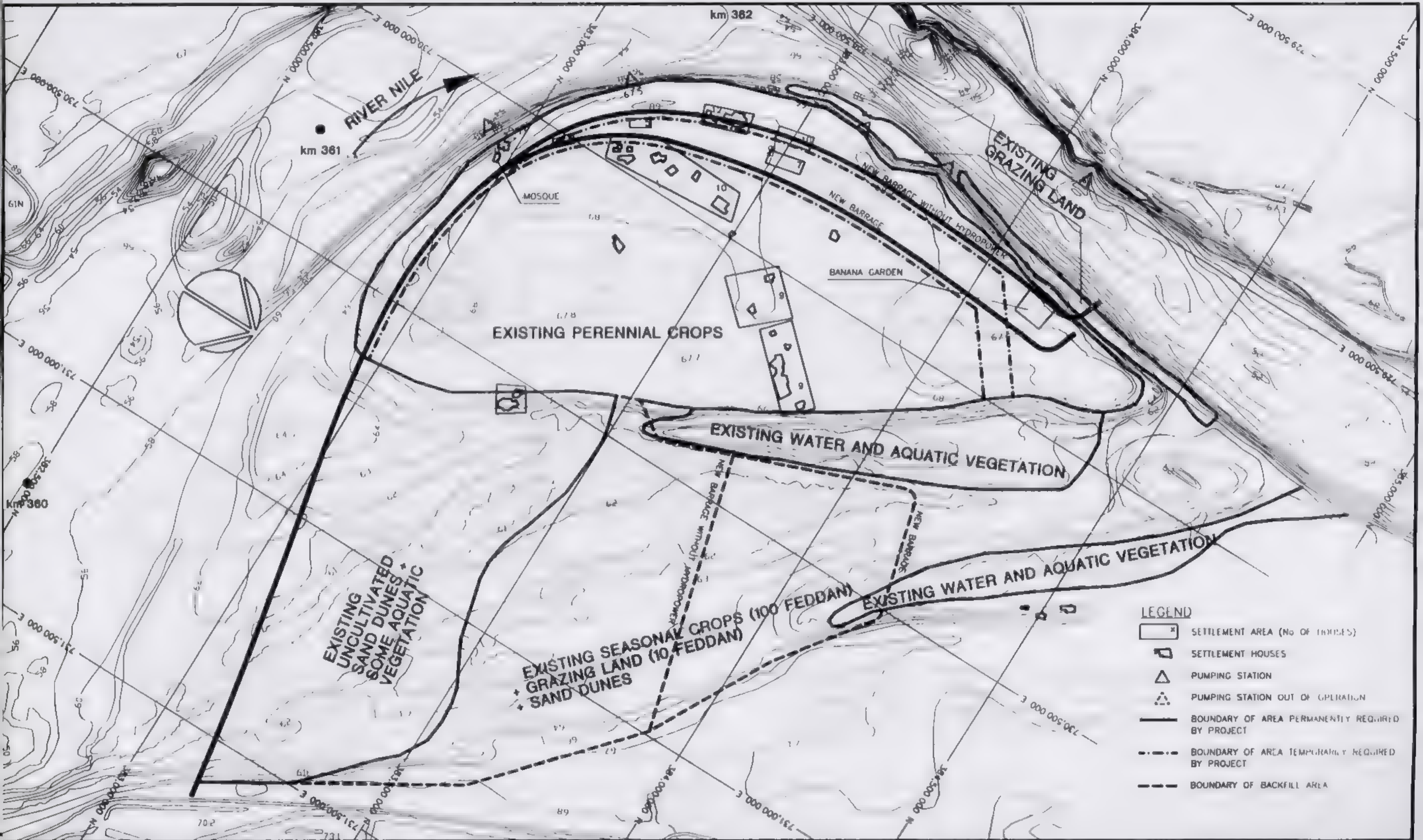

| | | | |
|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | NAGA HAMMADI BARRAGE DEVELOPMENT | |
| Lahmeyer | Electrowatt | FEASIBILITY STUDY | JANUARY 1997 |
| Sogreah | A. C. E. | UNIFIED POWER SYSTEM | FIGURE 6-3 |



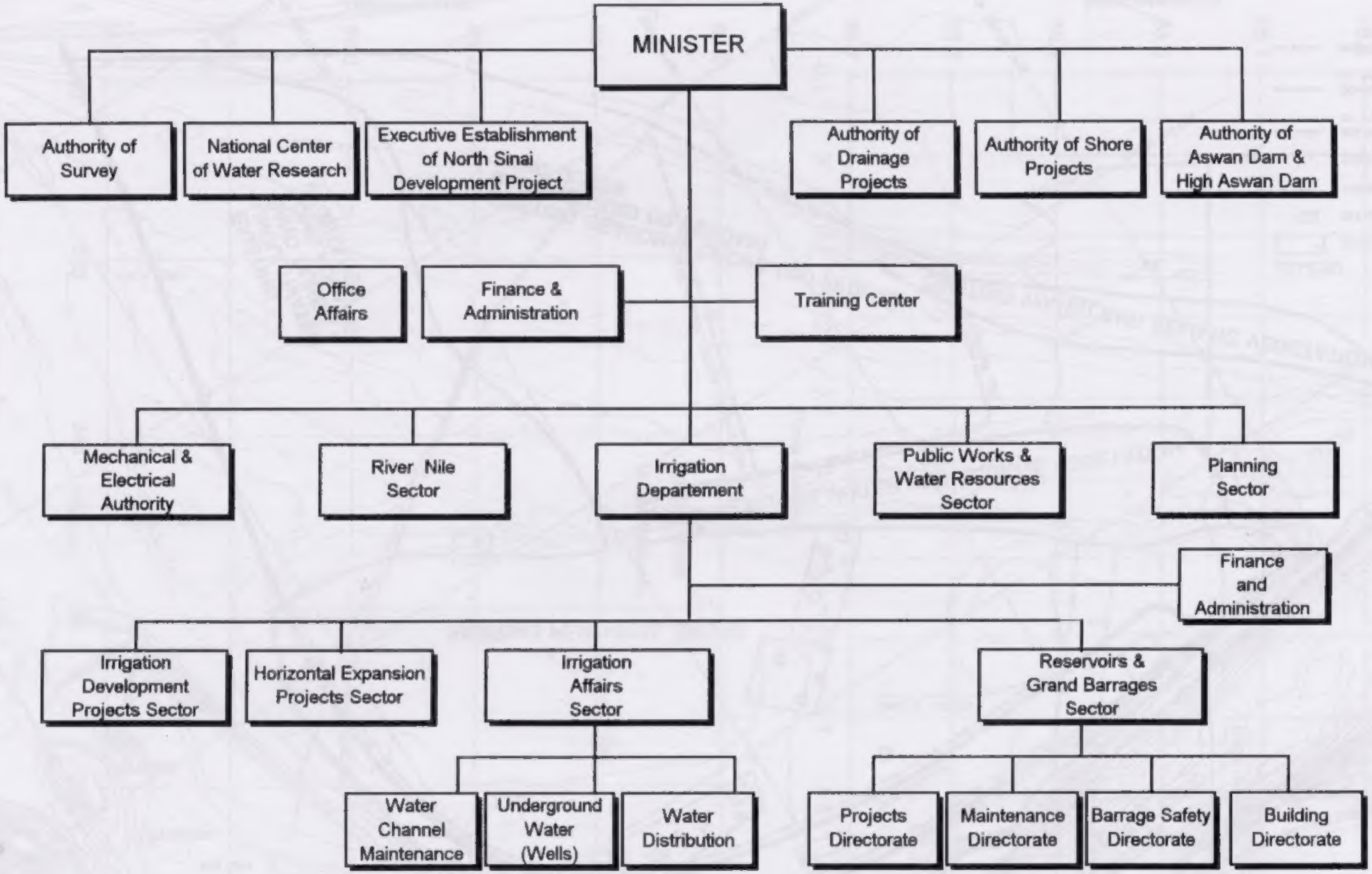




Steady state flow : 1842 $m^3/s$

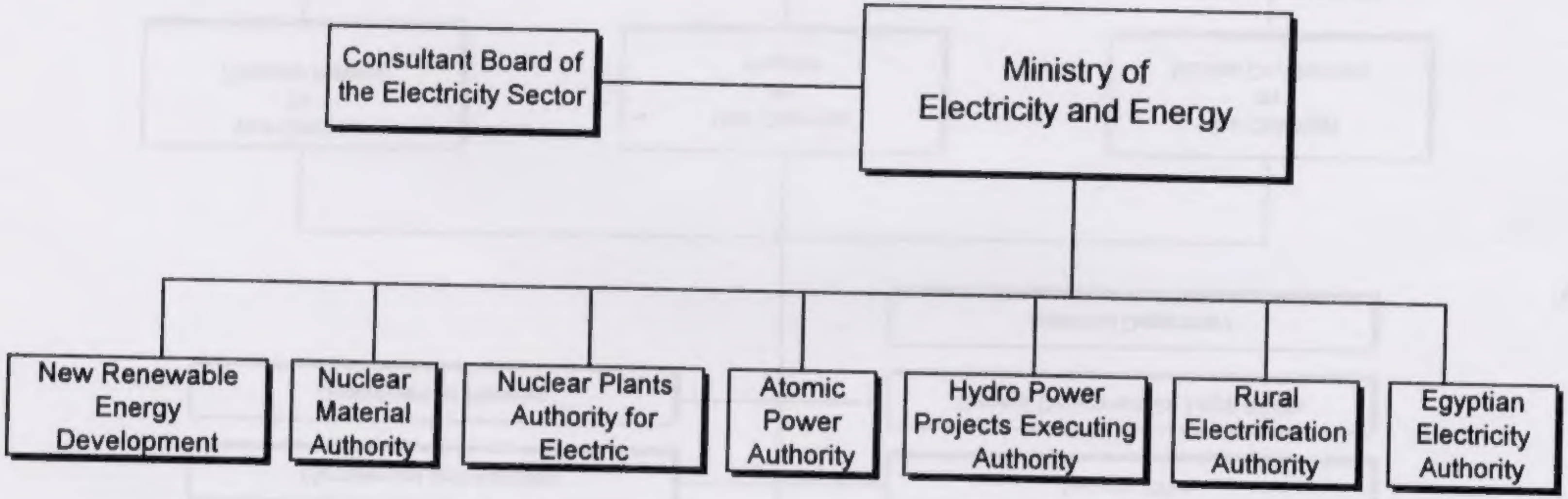
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | NAGA HAMMADI BARRAGE DEVELOPMENT | |
|---|---|---|---|
| Lahmeyer | Electrowatt | FEASIBILTY STUDY | JANUARY 1997 |
| Sogreah | A-C-E | U/S WATER LEVELS DUE TO LOAD REJECTION | FIGURE 6-4 |



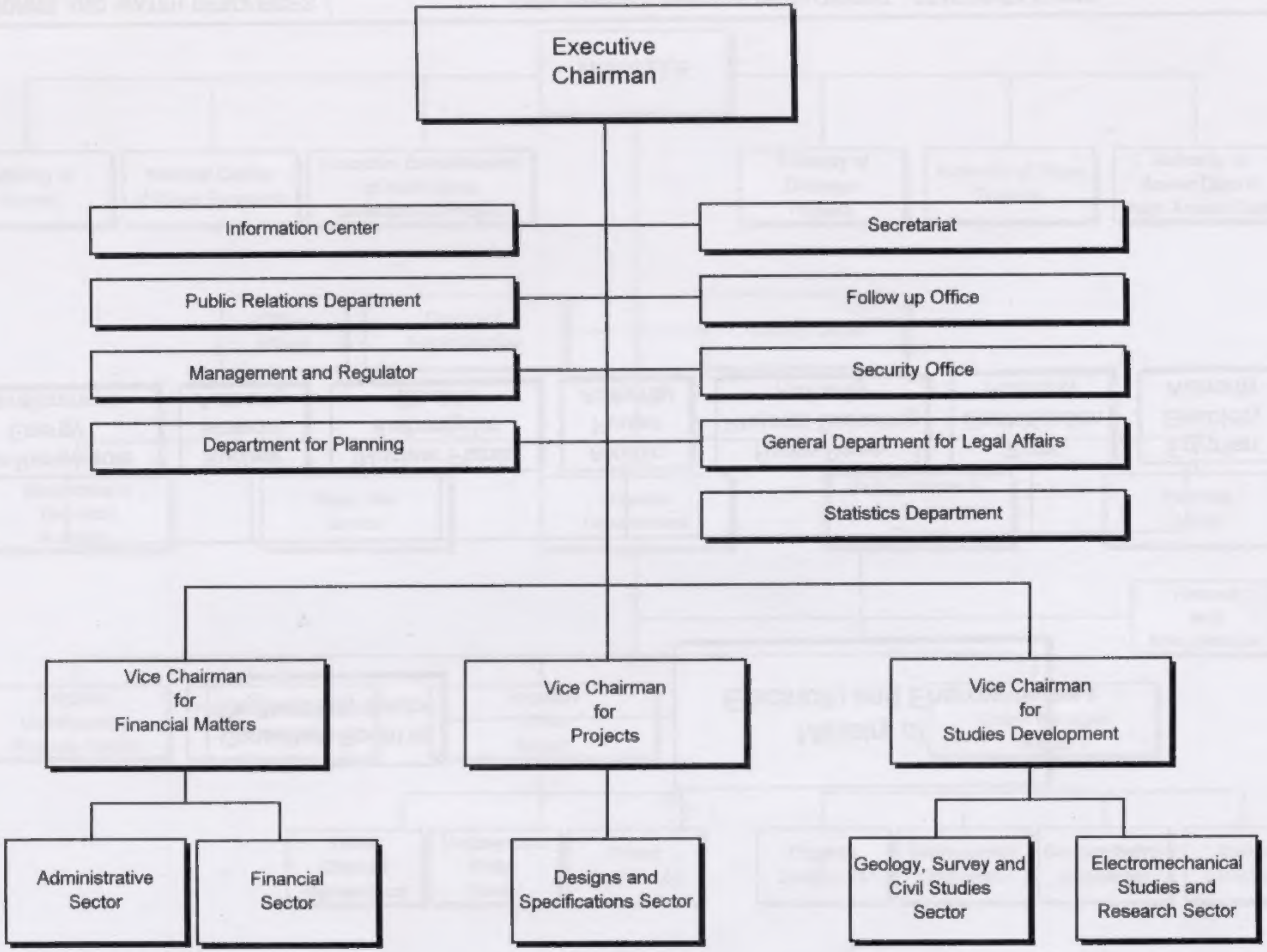

| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | IMPACT MAP - DOM ISLAND AND FLOOD CHANNEL | FIGURE 7-1 |




| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | ORGANIZATION CHART OF THE MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | FIGURE 8-2 |




| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
|---|---|---|---|---|---|
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | ORGANIZATION CHART OF THE EGYPTIAN ELECTRICITY SECTOR | FIGURE 8-3 |




| | | | | | |
|---|---|---|---|---|---|
| MINISTRY OF PUBLIC WORKS AND WATER RESOURCES | | | | NAGA HAMMADI BARRAGE DEVELOPMENT - FEASIBILITY STUDY | JANUARY 1997 |
| Lahmeyer | Electrowatt | Sogreah | A. C. E. | ORGANIZATION CHART OF THE HYDROPOWER PROJECTS EXECUTING AUTHORITY (HPPEA) | FIGURE 8-4 |

rehabilitation works was then developed within the Interim Study. This highlighted a number of critical factors related to:

- the success of the compaction grouting of the foundation soil,
- the technical difficulties of grouting works between the piers and the uncertainties of being able to extend the grouting under the apron slab.
- the relatively high apron slab and adjacent new rip-rap counterweir on which energy dissipation would take place during low tailwater levels.
- the rehabilitation of the 66-year-old gates, which appears to be possible now. However, if this would be the case in another 40 years (30 years after 2005) cannot be predicted with any certainty.

In discussions with the POE following the presentation of the assessment of the Barrage conditions and a possible rehabilitation program, the panel concurred that Barrage rehabilitation appears to be technically viable but would involve considerable risks. Whilst a reasonable number of geotechnical and construction material investigations were carried out overall, it can never be determined with certainty that these encompass the full range of site conditions. Hence, there remains a risk in the reliability of some of the basic measures of rehabilitation resulting in an uncertain remaining lifetime. In other words, although the rehabilitation measures are technically viable, it is not possible to guarantee the function of the Barrage for a reasonably long period which is ultimately the basis on which it would be judged an acceptable option. The rehabilitation option of the existing Barrage was therefore discarded from further consideration.

1.4.2 New Barrage with Hydropower

During the Conceptual Study, an appropriate layout of a New Barrage with hydropower was identified with the axis on the downstream river bend of the River Nile around Geziret el Dom, at river km 362.49. Studies of the layout at that time could not be supported by geotechnical investigations, and hence were inconclusive regarding the technical viability of this layout. Therefore, it was decided to continue with geotechnical investigations in the larger area of the New Barrage layout.

In the course of the engineering evaluation, different options for the construction pit and diversion scheme were investigated:

(i) Construction pit for all concrete structures located within the riverbed, diverting the river flow through a side canal around the construction pit. The diversion canal would be either on the left bank of the river or to the right on El Dom Island.

(ii) Construction pit on El Dom Island. The existing river channel to the left of the island would remain unaffected as a result of construction. Following completion of the concrete structures of the New Barrage, the river channel would be relocated through the New Barrage. The original river bend to the left of El Dom Island would be permanently closed by an embankment dam.

Geotechnical investigations, including drilling and soil sampling, undertaken at both alternative locations for the New Barrage revealed a continuous clay layer at about 35 to 45 m depth below present river levels exists only in the area of option(i) below the river bend and adjacent left (western) bank, but not at site (ii) on the island.